[illegible] mortar is ~ 30 % of the rubble masonry
1 cub. yard takes ~ 1,7 barrels cement
and ~ 0,75 yards^3 sand
[illegible] masonry + 0,65 = barrels cement [illegible]
[illegible] sand 5 crushed [illegible]
[illegible]

Concrete-floor 2.15 bars. cement per $\overline{100}^2$ ft

For laying one perch stone allow 75 c for mortar
consisting of 1. Louisville, 3 lime-paste.
cost of labor and sand included
1 barr. lime lays 2 ½ perch stone
1 barr. sand weighs = ~ 300 lbs.
1 barr. = 4 ft^3

[redacted]

Wätten-tätning for murbruk

1) såpa - 25 quart | Alun - 10 lbs
watten 1 barrel | watten 1 barrel

Använd såp-lösningen först

2) 4 gall watten
1 lbs borax
4 lbs soda
½ lbs Castile tvål

meeting

[redacted]

THE

ARCHITECT'S AND BUILDER'S POCKET-BOOK

OF

MENSURATION, GEOMETRY. GEOMETRICAL PROBLEMS, TRIGONOMETRICAL FORMULAS AND TABLES. STRENGTH AND STABILITY OF FOUNDATIONS, WALLS. BUTTRESSES, PIERS, ARCHES, POSTS, TIES, BEAMS, GIRDERS, TRUSSES, FLOORS, ROOFS, ETC.

IN ADDITION TO WHICH IS

A GREAT AMOUNT OF CONDENSED INFORMATION:

STATISTICS AND TABLES RELATING TO CARPENTRY, MASONRY, DRAINAGE, PAINTING AND GLAZING, PLUMBING, PLASTERING, ROOFING, HEATING AND VENTILATION, WEIGHTS OF MATERIALS, CAPACITY AND DIMENSIONS OF NOTED CHURCHES, THEATRES, DOMES, TOWERS, SPIRES, ETC.,

WITH A GREAT VARIETY OF MISCELLANEOUS INFORMATION.

BY
FRANK EUGENE KIDDER, C.E., PH.D.,
CONSULTING ARCHITECT, DENVER, COLO.

ILLUSTRATED WITH OVER 500 ENGRAVINGS, MOSTLY FROM ORIGINAL DESIGNS.

TWELFTH EDITION,

REVISED AND GREATLY ENLARGED.

INCLUDING A GLOSSARY OF TECHNICAL TERMS—ANCIENT AND MODERN.

FIRST THOUSAND.

NEW YORK:
JOHN WILEY & SONS,
53 EAST TENTH STREET.
19

Press of J. J. Little & Co.,
Astor Place, New York.

This Book

IS RESPECTFULLY DEDICATED TO THOSE WHOSE KINDNESS HAS ENABLED ME TO PRODUCE IT.

TO MY PARENTS,

WHO GAVE ME THE EDUCATION UPON WHICH IT IS BASED;

TO MY WIFE,

FOR HER LOVING SYMPATHY, ENCOURAGEMENT, AND ASSISTANCE;

TO ORLANDO W. NORCROSS

OF WORCESTER, MASS.,

WHOSE SUPERIOR PRACTICAL KNOWLEDGE OF ALL THAT PERTAINS TO BUILDING HAS GIVEN ME A MORE INTELLIGENT AND PRACTICAL VIEW OF THE SCIENCE OF CONSTRUCTION THAN I SHOULD OTHERWISE HAVE OBTAINED.

TWELFTH EDITION.

THE following revisions and additions have been made in this edition:

The chapter on Fire-proof Floors has been entirely rewritten and extended to conform to present practice, and several pages of revisions and additions have been made in Chapter XXV.

Several pages of tables relating to iron beams have been omitted, and other tables substituted in their place. New tables have been added in Chapter XI., giving the strength of H-shaped and rectangular cast-iron columns, and of the new "Gray" steel column. A special article on the Strength of Cast-iron Bearing Plates has been added to Chapter X., and new tables are given in Chapter VI. for the Strength of Masonry.

There are also several changes in Part III., particularly a revision of the article on Steam-heating, and several new pages giving the cost per square and cubic foot of public and private buildings. Altogether there are about one hundred pages of revised and new matter in this edition.

F. E. KIDDER.

DENVER, *March* 1, 1895.

PREFACE TO THE NINTH EDITION.

WITHIN the past four years the introduction of steel in building construction has been so rapid, and the changes thereby occasioned in the tables relating to the strength of materials so great, that it became necessary to revise all that portion of the book which relates to iron and steel construction. After undertaking this revision, it was found that the changes would be so great as to necessitate resetting a large portion of the book, and the author then decided to improve the opportunity to rearrange Part III., and to make certain additions thereto that he has had in contemplation for some time. The present edition, therefore, is largely a new book, all of Chapters XXIII. and XXV., and nearly all of Chapters XI., XIII., and XIV., being rewritten, and one hundred pages of new matter added in the second part alone.

Part III. has been rearranged and enlarged by about eighty pages of miscellaneous information of especial value to architects, and a glossary of sixty pages added as an appendix.

The new matter contained on pages 746-773, it is believed, will be of especial interest to architects and draughtsmen, as the data there given are not readily accessible elsewhere. It will be noticed that in the list of Noted American Architects there are many dates wanting; if such readers as may be able to supply them will kindly inform the author, he will be greatly obliged.

The author is always pleased to receive criticism and suggestions, and is ever willing to give further explanation of any portion of the book that may not be readily understood.

F. E. KIDDER.

DENVER, COL., *November* 3, 1891.

PREFACE.

In preparing the following pages, it has ever been the aim of the author to give to the architects and builders of this country a *reference book* which should be for them what Trautwine's "Pocket-Book" is to engineers, — a compendium of practical facts, rules, and tables, presented in a form as convenient for application as possible, and as reliable as our present knowledge will permit. Only so much *theory* has been given as will render the application of the formulas more apparent, and aid the student in understanding, in some measure, the principles upon which the formulas are based. It is believed that nothing has been given in this book but what has been borne out in practice.

As this book was not written for *engineers*, the more intricate problems of building construction, which may fairly be said to come within the province of the civil engineer, have been omitted.

Desiring to give as much information as possible likely to be of service to architects and builders, the author has borrowed and quoted from many sources, in most cases with the permission of the authors. Much practical information has been derived from the various handbooks published by the large manufacturers of rolled-iron beams, bars, etc.; and the author has always found the publishers willing to aid him whenever requested.

Although but very little has been taken from Trautwine's "Pocket-Book for Engineers," yet this valuable book has served the author as a model, which he has tried to imitate as well as the difference in the subjects would permit; and if his work shall prove of as much value to architects and builders as Mr. Trautwine's has to engineers, he will feel amply rewarded for his labor.

As it is impossible for the author to verify all of the dimensions and miscellaneous information contained in Part III., he cannot speak for their accuracy, except that they were in all cases taken from what were considered reliable sources of information. The tables in Part II. have been carefully computed, and it is believed are free from any large errors. There are so many points of information often required by architects and builders, that it is difficult for one person to compile them all; and although the present volume is by no means a small one, yet the author desires to make his work as useful as possible to those for whom it has been prepared, and he will therefore be pleased to receive any information of a serviceable nature pertaining to architecture or building, that it may be inserted in future editions should such become necessary, and for the correction of any errors that may be found.

The author, while compiling this volume, has consulted a great number of works relating to architecture and building; and as he has frequently been asked by students and draughtsmen to refer them to books from which they might acquire a better knowledge of construction and building, the following list of books is given as valuable works on the various subjects indicated by the titles: —

"Notes on Building Construction," compiled for the use of the students in the science and art schools, South Kensington, England. 3 vols. Rivingtons, publishers, London.

"Building Superintendence," by T. M. Clark, architect and professor of architecture, Massachusetts Institute of Technology. J. R. Osgood & Co., publishers, Boston.

"The American House Carpenter" and "The Theory of Transverse Strains," both by Mr. R. G. Hatfield, architect, formerly of New York.

"Graphical Analysis of Ro Trusses," by Professor Charles E. Green of the University of chigan.

"The Fire Protection of ' by C. J. H. Woodbury, inspector for the Factory urance Companies. John Wiley & Sons, publisl

"House Drainage and Water Service," by James C. Bayles, editor of "The Iron Age" and "The Metal Worker." David Williams, publisher, New York.

"The Builders' Guide and Estimators' Price-Book," and "Plaster and Plastering, Mortars, and Cements," by Fred. T. Hodgson, editor of "The Builder and Wood Worker." Industrial Publication Company, New York.

"Foundations and Concrete Works" and "Art of Building," by E. Dobson. Weale's Series, London.

It would be well if all of the above books might be found in every architect's office; but if the expense prevents that, the ambitious student and draughtsman should at least make himself acquainted with their contents. These works will also be found of great value to the enterprising builder.

PREFACE TO THE FOURTH EDITION.

It is now a little more than two years since "The Architect's and Builder's Pocket-Book" was first introduced to the public. During that time the author has received so many encouraging words and suggestions from a large number of architects and builders, that he desires to acknowledge their kindness, and to express the hope that the book will always merit their commendation.

When preparing the book for publication, especial care and thought were given to the second part of the book; trusting that, if once well done, it would need but little revision for a number of years. The first part, also, it is believed, is quite complete in its way. For Part III., however, the author found time merely to compile such matter as he believed to be of practical value to architects or builders, thinking that, should the book prove a success, this part could be easily revised and enlarged; and, since the second edition was published, the author has devoted such time as he could command to revising such portions as upon investigation seemed to require it, and preparing additional matter.

It is the intention of the author, seconded by the publishers, to make each edition of the book more complete and perfect than the one preceding, in the hope that it may in time become to the architects of the present day what Gwilt's "Encyclopædia" was to those of former days. The great diversity of information, however, required by an architect, or those having to do

of time to devote to the work, to make such a book as complete as could be desired.

In the Preface to the first edition it was requested that those who might have information or suggestions which would increase the value of the book would kindly send them to the author, or advise him of any errors that should be discovered.

Several persons generously replied to this invitation; and several small errors have been corrected, and additional information given, as the result. It is believed, however, that there are yet many who have thought, at times, of various ways in which the book could be improved, or have in their private note-books practical data or suggestions which others in the profession would be glad to possess; and it is hoped all such will feel it for the interest of the profession to forward such items to the author.

Any records or reports of tests of the strength of building materials of any kind will be especially appreciated.

To the list of books given in the former Preface the author would add the following, which have been of much assistance in the preparation of the pages on steam-heating, and in his professional practice: —

"The Principles of Heating and Ventilation, and their Practical Application," by John S. Billings, M.D., LL.D., Sanitary Engineer, New York.

"Steam-Heating for Buildings; or, Hints to Steam-Fitters, by William J. Baldwin, M.E. John Wiley & Sons, New York.

"Steam." Babcock & Wilcox Company, New York and Glasgow.

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PART I.

PRACTICAL

ARITHMETIC, GEOMETRY, AND TRIGONOMETRY.

RULES, TABLES, AND PROBLEMS

PRACTICAL

ARITHMETIC AND GEOMETRY.

SIGNS AND CHARACTERS.

The following signs and characters are generally used to denote and abbreviate the several mathematical operations:—

The sign $=$ means equal to, or equality.

$-$ means minus or less, or subtraction.

$+$ means plus, or addition.

$\times$ means multiplied by, or multiplication.

$\div$ means divided by, or division.

^{2} ^{3} Index or power, meaning that the number to which they are added is to be squared (^{2}) or cubed (^{3}).

: is to
:: so is — Signs of proportion.
: to

$\sqrt{}$ means that the square root of the number before which it is placed is required.

$\sqrt[3]{}$ means that the cube root of the number before which it is placed is required.

——— the *bar* indicates that all the numbers under it are to be taken together.

( ) the *parenthesis* means that all the numbers between are to be taken as one quantity.

. means decimal parts; thus, 2.5 means $2\frac{5}{10}$, 0.46 means $\frac{46}{100}$.

$^\circ$ means degrees, $'$ minutes, $''$ seconds.

$\therefore$ means hence.

INVOLUTION.

To square a number, multiply the number by itself, and the product will be the square; thus, the square of $18 = 18 \times 18 = 324$.

The cube of a number is the product obtained by multiplying the number by itself, and that product by the number again; thus, the cube of $14 = 14 \times 14 \times 14 = 2744$.

The fourth power of a number is the product obtained by multiplying the number by itself four times; thus, the fourth power of $10 = 10 \times 10 \times 10 \times 10 = 10000$.

EVOLUTION.

Square Root. — Rule for determining the square root of a number.

1st, Divide the given number into periods of two figures each, commencing at the right if it is a whole number, and at the decimal-point if there are decimals; thus, $\dot{1}0\dot{2}3\dot{6}.8\dot{1}2\dot{6}$.

2d, Find the largest square in the left-hand period, and place its root in the quotient; subtract the said square from the left-hand period, and to the remainder bring down the next period for a new dividend.

3d, Double the root already found, and annex one cipher for a trial divisor, see how many times it will go in the dividend, and put the number in the quotient; also, in place of the cipher in the divisor, multiply this final divisor by the number in the quotient just found, and subtract the product from the dividend, and to the remainder bring down the next period for a new dividend, and proceed as before. If it should be found that the trial divisor cannot be contained in the dividend, bring down the next period for a new dividend, and annex another cipher to the trial divisor, and put a cipher in the quotient, and proceed as before.

EXAMPLE. $\dot{1}0\dot{2}3\dot{6}.8\dot{1}2\dot{6}$ (101.17 square root.

```
          1
          -
    201 ) 0236
           201
          ----
   2021 ) 3581
          2021
          ------
  20227 ) 156026
          141589
          ------
           14437
```

Cube Root. — To extract the cube root of a number, point off the number from right to left into periods of three figures each, and, if there is a decimal, commence at the decimal-point, and point off into periods, going both ways.

Ascertain the highest root of the first period, and place to right of number, as in long division; cube the root thus found, and subfr[illegible] he first period; to the remainder annex the next period: [illegible]ne root already found, and multiply by three, and annex

two ciphers for the trial divisor. Find how often this trial divisor is contained in the dividend, and write the result in the root.

Add together the trial divisor, three times the product of the first figure of the root by the second with one cipher annexed, and the square of the second figure in the root; multiply the sum by the last figure in the root, and subtract from the dividend; to the remainder annex the next period, and proceed as before.

When the trial divisor is greater than the dividend, write a cipher in the root, annex the next period to the dividend, and proceed as before.

Desired the $\sqrt[3]{493039}$.

	493039 (79 cube root.
$7 \times 7 \times 7 =$	343
$7 \times 7 \times 3 =$ 14700	150039
$7 \times 9 \times 3 =$ 1890	
$9 \times 9 =$ 81	
16671	150039

Desired the $\sqrt[3]{403583.419}$.

	403583.419 (73.9 cube root.
$7 \times 7 \times 7 =$	343
$7 \times 7 \times 3 =$ 14700	60583
$7 \times 3 \times 3 =$ 630	
$3 \times 3 =$ 9	
15339	46017
$73 \times 73 \times 3 =$ 1598700	14566419
$73 \times 9 \times 3 =$ 19710	
$9 \times 9 =$ 81	
1618491	14566419

Desired the $\sqrt[3]{158252.632929}$.

	158252.632929 (54.09 cube root
$5 \times 5 \times 5 =$	125
$5 \times 5 \times 3 =$ 7500	33225
$5 \times 4 \times 3 =$ 600	
$4 \times 4 =$ 16	
8116	32464
$540 \times 540 \times 3 =$ 87480000	788632929
$540 \times 9 \times 3 =$ 145800	
$9 \times 9 =$ 81	
87625881	788632929

TABLE

OF

SQUARES, CUBES, SQUARE ROOTS, CUBE ROOTS, AND RECIPROCALS,

From 1 to 1054.

The following table, taken from Searle's "Field Engineering," will be found of great convenience in finding the square, cube, square root, cube root, and reciprocal of any number from 1 to 1054. The reciprocal of a number is the quotient obtained by dividing 1 by the number. Thus the reciprocal of 8 is $1 \div 8 = 0.125$.

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
1	1	1	1.0000000	1.0000000	1.000000000
2	4	8	1.4142136	1.2599210	.500000000
3	9	27	1.7320508	1.4422496	.333333333
4	16	64	2.0000000	1.5874011	.250000000
5	25	125	2.2360680	1.7099759	.200000000
6	36	216	2 4494897	1.8171206	.166666667
7	49	343	2.6457513	1.9129312	.142857143
8	64	512	2.8284271	2.0000000	.125000000
9	81	729	3.0000000	2.0800837	.111111111
10	100	1000	3.1622777	2.1544347	.100000000
11	121	1331	3.3166248	2.2239801	.090909091
12	144	1728	3.4641016	2.2894286	.083333333
13	169	2197	3.6055513	2.3513347	.076923077
14	196	2744	3.7416574	2.4101422	.071428571
15	225	3375	3.8729833	2.4662121	.066666667
16	256	4096	4.0000000	2.5198421	.062500000
17	289	4913	4.1231056	2.5712816	.058823529
18	324	5832	4.2426407	2.6207414	.055555556
19	361	6859	4.3588989	2.6684016	.052631579
20	400	8000	4.4721360	2.7144177	.050000000
21	441	9261	4.5825757	2.7589243	.047619048
22	484	10648	4.6904158	2.8020393	.045454545
23	529	12167	4.7958315	2.8438670	.043478261
24	576	13824	4.8989795	2.8844991	.041666667
25	625	15625	5.0000000	2.9240177	.040000000
26	676	17576	5.0990195	2.9624960	.038461538
27	729	19683	5.1961524	3.0000000	.037037037
28	784	21952	5.2915026	3.0365889	.035714286
29	841	24389	5.3851648	3.0723168	.034482759
30	900	27000	5.4772256	3.1072325	.033333333
31	961	29791	5.5677644	3.1413806	.032258065
32	1024	32768	5.6568542	3.1748021	.031250000
33	1089	35937	5.7445626	3.2075343	030303030
34	1156	39304	5.8309519	3.2396118	.029411765
35	1225	42875	5.9160798	3.2710663	.028571429
36	1296	46656	6.0000000	3.3019272	.027777778
37	1369	50653	6.0827625	3.3322218	.027027027
38	1444	54872	6.1644140	3.3619754	.026315789
39	1521	59319	6.2449980	3.3912114	.025641026
40	1600	64000	6.3245553	3.4199519	.025000000
41	1681	68921	6.4031242	3.4482172	.024390244
42	1764	74088	6.4807407	3.4760266	.023809524
43	1849	79507	6.5574385	3.5033981	.023255814
44	1936	85184	6.6332496	3.5303483	.022727273
45	2025	91125	6.7082039	3.5568933	.022222222
46	2116	97336	6.7823300	3.5830479	.021739130
47	2209	103823	6.8556546	3.6088261	.021276600
48	2304	110592	6.9282032	3.6342411	.020833333
49	2401	117649	7.0000000	3.6593057	.020408163
50	2500	125000	7.0710678	3.6840314	.020000000
51	2601	132651	7.1414284	3.7084298	.019607843
52	2704	140608	7.2111026	3.7325111	.019230769
53	2809	148877	7.2801099	3.7562858	.018867925
54	2916	157464	7.3484692	3.7797631	.018518519
55	3025	166375	7.4161985	3.8029525	.018181818
56	3136	175616	7.4833148	3.8258624	.017857143
57	3249	185193	7.5498344	3.8485011	.017543860
58	3364	195112	7.6157731	3.8708766	.017241379
59	3481	205379	7.6811457	3.8929965	.016949153
	3600	216000	7.7459667	3.9148676	.016666667
	3721	226981	7.8102497	3.9364972	.016393443
	3844	238328	7.8740079	3.9578915	016129032

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
63	3969	250047	7.9372539	3.9790571	.015873016
64	4096	262144	8.0000000	4.0000000	.015625000
65	4225	274625	8.0622577	4.0207256	.015384615
66	4356	287496	8.1240384	4.0412401	.015151515
67	4489	300763	8.1853528	4.0615480	.014925373
68	4624	314432	8.2462113	4.0816551	.014705882
69	4761	328509	8.3066239	4.1015661	.014492754
70	4900	343000	8.3666003	4.1212853	.014285714
71	5041	357911	8.4261498	4.1408178	.014084507
72	5184	373248	8.4852814	4.1601676	.013888889
73	5329	389017	8.5440037	4.1793390	.013698630
74	5476	405224	8.6023253	4.1983364	.013513514
75	5625	421875	8.6602540	4.2171633	.013333333
76	5776	438976	8.7177979	4.2358236	.013157895
77	5929	456533	8.7749644	4.2543210	.012987013
78	6084	474552	8.8317609	4.2726586	.012820513
79	6241	493039	8.8881944	4.2908404	.012658228
80	6400	512000	8.9442719	4.3088695	.012500000
81	6561	531441	9.0000000	4.3267487	.012345679
82	6724	551368	9.0553851	4.3444815	.012195122
83	6889	571787	9.1104336	4.3620707	.012048193
84	7056	592704	9.1651514	4.3795191	011904762
85	7225	614125	9.2195445	4.3968296	.011764706
86	7396	636056	9.2736185	4.4140049	.011627907
87	7569	658503	9.3273791	4.4310476	.011494253
88	7744	681472	9.3808315	4.4479602	.011363636
89	7921	704969	9.4339811	4.4647451	.011235955
90	8100	729000	9.4868330	4.4814047	.011111111
91	8281	753571	9.5393920	4.4979414	.010989011
92	8464	778688	9.5916630	4.5143574	.010869565
93	8649	804357	9.6436508	4.5306549	.010752688
94	8836	830584	9.6953597	4.5468359	.010638298
95	9025	857375	9.7467943	4.5629026	.010526316
96	9216	884736	9.7979590	4.5788570	.010416667
97	9409	912673	9.8488578	4.5947009	.010309278
98	9604	941192	9.8994949	4.6104363	.010204082
99	9801	970299	9.9498744	4.6260650	.010101010
100	10000	1000000	10.0000000	4.6415888	.010000000
101	10201	1030301	10.0498756	4.6570095	.009900990
102	10404	1061208	10.0995049	4.6723287	.009803922
103	10609	1092727	10.1488916	4.6875482	.009708738
104	10816	1124864	10.1980390	4.7026694	.009615385
105	11025	1157625	10.2469508	4.7176940	.009523810
106	11236	1191016	10.2956301	4.7326235	.009433962
107	11449	1225043	10.3440804	4.7474594	.009345794
108	11664	1259712	10.3923048	4.7622032	.009259259
109	11881	1295029	10.4403065	4.7768562	.009174312
110	12100	1331000	10.4880885	4.7914199	.009090909
111	12321	1367631	10.5356538	4.8058955	.009009009
112	12544	1404928	10.5830052	4.8202845	.008928571
113	12769	1442897	10.6301458	4.8345881	.008849558
114	12996	1481544	10.6770783	4.8488076	.008771930
115	13225	1520875	10.7238053	4.8629442	.008695652
116	13456	1560896	10.7703296	4.8769990	.008620690
117	13689	1601613	10.8166538	4.8909732	.008547009
118	13924	1643032	10.8627805	4.9048681	.008474576
119	14161	1685159	10.9087121	4.9186847	.008403361
120	14400	1728000	10.9544512	4.9324242	.008333333
121	14641	1771561	11.0000000	4.9460874	.008264463
122	14884	1815848	11.0453610	4.9596757	.008196721
123	15129	1860867	11.0905365	4.9731898	.008130081
124	15376	1906624	11.1355287	4.9866310	.008064516

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
125	15625	1953125	11.1803399	5.0000000	.008000000
126	15876	2000376	11.2249722	5.0132979	.007936508
127	16129	2048383	11.2694277	5.0265257	.007874016
128	16384	2097152	11.3137085	5.0396842	.007812500
129	16641	2146689	11.3578167	5.0527743	.007751938
130	16900	2197000	11.4017543	5.0657970	.007692308
131	17161	2248091	11.4455231	5.0787531	.007633588
132	17424	2299968	11.4891253	5.0916434	.007575758
133	17689	2352637	11.5325626	5.1044687	.007518797
134	17956	2406104	11.5758369	5.1172299	.007462687
135	18225	2460375	11.6189500	5.1299278	.007407407
136	18496	2515456	11.6619038	5.1425632	.007352941
137	18769	2571353	11.7046999	5.1551367	.007299270
138	19044	2628072	11.7473401	5.1676493	.007246377
139	19321	2685619	11.7898261	5.1801015	.007194245
140	19600	2744000	11.8321596	5.1924941	.007142857
141	19881	2803221	11.8743421	5.2048279	.007092199
142	20164	2863288	11.9163753	5.2171034	.007042254
143	20449	2924207	11.9582607	5.2293215	.006993007
144	20736	2985984	12.0000000	5.2414828	.006944444
145	21025	3048625	12.0415946	5.2535879	.006896552
146	21316	3112136	12.0830460	5.2656374	.006849315
147	21609	3176523	12.1243557	5.2776321	.006802721
148	21904	3241792	12.1655251	5.2895725	.006756757
149	22201	3307949	12.2065556	5.3014592	.006711409
150	22500	3375000	12.2474487	5.3132928	.006666667
151	22801	3442951	12.2882057	5.3250740	.006622517
152	23104	3511808	12.3288280	5.3368033	.006578947
153	23409	3581577	12.3693169	5.3484812	.006535948
154	23716	3652264	12.4096736	5.3601084	.006493506
155	24025	3723875	12.4498996	5.3716854	.006451613
156	24336	3796416	12.4899960	5.3832126	.006410256
157	24649	3869893	12.5299641	5.3946907	.006369427
158	24964	3944312	12.5698051	5.4061202	.006329114
159	25281	4019679	12.6095202	5.4175015	.006289308
160	25600	4096000	12.6491106	5.4288352	.006250000
161	25921	4173281	12.6885775	5.4401218	.006211180
162	26244	4251528	12.7279221	5.4513618	.006172840
163	26569	4330747	12.7671453	5.4625556	.006134969
164	26896	4410944	12.8062485	5.4737037	.006097561
165	27225	4492125	12.8452326	5.4848066	.006060606
166	27556	4574296	12.8840987	5.4958647	.006024096
167	27889	4657463	12.9228480	5.5068784	.005988024
168	28224	4741632	12.9614814	5.5178484	.005952381
169	28561	4826809	13.0000000	5.5287748	.005917160
170	28900	4913000	13.0384048	5.5396583	.005882353
171	29241	5000211	13.0766968	5.5504991	.005847953
172	29584	5088448	13.1148770	5.5612978	.005813953
173	29929	5177717	13.1529464	5.5720546	.005780347
174	30276	5268024	13.1909060	5.5827702	.005747126
175	30625	5359375	13.2287566	5.5934447	.005714286
176	30976	5451776	13.2664992	5.6040787	.005681818
177	31329	5545233	13.3041347	5.6146724	.005649718
178	31684	5639752	13.3416641	5.6252263	.005617978
179	32041	5735339	13.3790882	5.6357408	.005586592
180	32400	5832000	13.4164079	5.6462162	.005555556
181	32761	5929741	13.4536240	5.6566528	.005524862
182	33124	6028568	13.4907376	5.6670511	.005494505
183	33489	6128487	13.5277493	5.6774114	.005464481
184	33856	6229504	13.5646600	5.6877340	.005434783
185	34225	6331625	13.6014705	5.6980192	.005405405
186	34596	6434856	13.6381817	5.7082675	.005376344

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
187	34969	6539203	13.6747943	5.7184791	.005347594
188	35344	6644672	13.7113092	5.7286543	.005319149
189	35721	6751269	13.7477271	5.7387936	.005291005
190	36100	6859000	13.7840488	5.7488971	.005263158
191	36481	6967871	13.8202750	5.7589652	.005235602
192	36864	7077888	13.8564065	5.7689982	.005208333
193	37249	7189057	13.8924440	5.7789966	.005181347
194	37636	7301384	13.9283883	5.7889604	.005154639
195	38025	7414875	13.9642400	5.7988900	.005128205
196	38416	7529536	14.0000000	5.8087857	.005102041
197	38809	7645373	14.0356688	5.8186479	.005076142
198	39204	7762392	14.0712473	5.8284767	.005050505
199	39601	7880599	14.1067360	5.8382725	.005025126
200	40000	8000000	14.1421356	5.8480355	.005000000
201	40401	8120601	14.1774469	5.8577660	.004975124
202	40804	8242408	14.2126704	5.8674643	.004950495
203	41209	8365427	14.2478068	5.8771307	.004926108
204	41616	8489664	14.2828569	5.8867653	.004901961
205	42025	8615125	14.3178211	5.8963685	.004878049
206	42436	8741816	14.3527001	5.9059406	.004854369
207	42849	8869743	14.3874946	5 9154817	.004830918
208	43264	8998912	14.4222051	5 9249921	.004807692
209	43681	9129329	14.4568323	5.9344721	.004784689
210	44100	9261000	14.4913767	5.9439220	.004761905
211	44521	9393931	14.5258390	5.9533418	.004739336
212	44944	9528128	14.5602198	5.9627320	.004716981
213	45369	9663597	14.5945195	5.9720926	.004694836
214	45796	9800344	14.6287388	5.9814240	.004672897
215	46225	9938375	14.6628783	5.9907264	.004651163
216	46656	10077696	14.6969385	6.0000000	.004629630
217	47089	10218313	14.7309199	6.0092450	.004608295
218	47524	10360232	14.7648231	6.0184617	.004587156
219	47961	10503459	14.7986486	6.0276502	.004566210
220	48400	10648000	14.8323970	6.0368107	.004545455
221	48841	10793861	14.8660687	6.0459435	.004524887
222	49284	10941048	14.8996644	6.0550489	.004504505
223	49729	11089567	14.9331845	6.0641270	.004484305
224	50176	11239424	14.9666295	6.0731779	.004464286
225	50625	11390625	15.0000000	6.0822020	.004444444
226	51076	11543176	15.0332964	6.0911994	.004424779
227	51529	11697083	15.0665192	6.1001702	.004405286
228	51984	11852352	15.0996689	6.1091147	.004385965
229	52441	12008989	15.1327460	6.1180332	.004366812
230	52900	12167000	15.1657509	6.1269257	.004347826
231	53361	12326391	15.1986842	6.1357924	.004329004
232	53824	12487168	15.2315462	6.1446337	.004310345
233	54289	12649337	15.2643375	6.1534495	.004291845
234	54756	12812904	15.2970585	6.1622401	.004273504
235	55225	12977875	15.3297097	6.1710058	.004255319
236	55696	13144256	15.3622915	6.1797466	.004237288
237	56169	13312053	15.3948043	6.1884628	.004219409
238	56644	13481272	15.4272486	6.1971544	.004201681
239	57121	13651919	15.4596248	6.2058218	.004184100
240	57600	13824000	15.4919334	6.2144650	.004166667
241	58081	13997521	15.5241747	6 2230843	.004149378
242	58564	14172488	15.5563492	6.2316797	.004132231
243	59049	14348907	15.5884573	6.2402515	.004115226
244	59536	14526784	15.6204994	6.2487998	.004098361
245	60025	14706125	15.6524758	6.2573248	.004081633
246	60516	14886936	15.6843871	6.2658266	.004065041
247	61009	15069223	15.7162336	6.2743054	.004048583
248	61504	15252992	15.7480157	6.2827613	.004032258

No.	Square Roots.	Cube Roots.	Reciprocals.
249	15.7797338	6.2911946	.004016064
250	15.8113883	6.2996053	.004000000
251	15.8429795	6.3079935	.003984064
252	15.8745079	6.3163596	.003968254
253	15.9059737	6.3247035	.003952569
254	15.9373775	6.3330256	.003937008
255	15.9687194	6.3413257	.003921569
256	16.0000000	6.3496042	.003906250
257	16.0312195	6.3578611	.003891051
258	16.0623784	6.3660968	.003875969
259	16.0934769	6.3743111	.003861004
260	16.1245155	6.3825043	.003846154
261	16.1554944	6.3906765	.003831418
262	16.1864141	6.3988279	.003816794
263	16.2172747	6.4069585	.003802281
264	16.2480768	6.4150687	.003787879
265	16.2788206	6.4231583	.003773585
266	16.3095064	6.4312276	.003759398
267	16.3401346	6.4392767	.003745318
268	16.3707055	6.4473057	.003731343
269	16.4012195	6.4553148	.003717472
270	16.4316767	6.4633041	.003703704
271	16.4620776	6.4712736	.003690037
272	16.4924225	6.4792236	.003676471
273	16.5227116	6.4871541	.003663004
274	16.5529454	6.4950653	.003649635
275	16.5831240	6.5029572	.003636364
276	16.6132477	6.5108300	.003623188
277	16.6433170	6.5186839	.003610108
278	16.6733320	6.5265189	.003597122
279	16.7032931	6.5343351	.003584229
280	16.7332005	6.5421326	.003571429
281	16.7630546	6.5499116	.003558719
282	16.7928556	6.5576722	.003546099
283	16.8226038	6.5654144	.003533569
284	16.8522995	6.5731385	.003521127
285	16.8819430	6.5808443	.003508772
286	16.9115345	6.5885323	.003496503
287	16.9410743	6.5962023	.003484321
288	16.9705627	6.6038545	.003472222
289	17.0000000	6.6114890	.003460208
290	17.0293864	6.6191060	.003448276
291	17.0587221	6.6267054	.003436426
292	17.0880075	6.6342874	.003424658
293	17.1172428	6.6418522	.003412960
294	17.1464282	6.6493998	.003401361
295	17.1755640	6.6569302	.003389831
296	17.2046505	6.6644437	.003378378
297	17.2336879	6.6719403	.003367003
298	17.2626765	6.6794200	.003355705
299	17.2916165	6.6868831	.003344482
300	17.3205081	6.6943295	.003333333
301	17.3493516	6.7017593	.003322259
302	17.3781472	6.7091729	.003311258
303	17.4068952	6.7165700	.003300330
304	17.4355958	6.7239508	.003289474
305	17.4642492	6.7313155	.003278689
306	17.4928557	6.7386641	.003267974
307	17.5214155	6.7459967	.003257329
308	17.5499288	6.7533134	.003246753
309	17.5783958	6.7606143	.003236246
310	17.6068169	6.7678995	.003225806

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
311	96721	30080231	17.6351921	6.7751690	.003215434
312	97344	30371328	17.6635217	6.7824229	.003205128
313	97969	30664297	17.6918060	6.7896613	.003194888
314	98596	30959144	17.7200451	6.7968844	.003184713
315	99225	31255875	17.7482393	6.8040921	.003174603
316	99856	31554496	17.7763888	6.8112847	.003164557
317	100489	31855013	17.8044938	6.8184620	.003154574
318	101124	32157432	17.8325545	6.8256242	.003144654
319	101761	32461759	17.8605711	6.8327714	.003134796
320	102400	32768000	17.8885438	6.8399037	.003125000
321	103041	33076161	17.9164729	6.8470213	.003115265
322	103684	33386248	17.9443584	6.8541240	.003105590
323	104329	33698267	17.9722008	6.8612120	.003095975
324	104976	34012224	18.0000000	6.8682855	.003086420
325	105625	34328125	18.0277564	6.8753443	.003076923
326	106276	34645976	18.0554701	6.8823888	.003067485
327	106929	34965783	18.0831413	6.8894188	.003058104
328	107584	35287552	18.1107703	6.8964345	.003048780
329	108241	35611289	18.1383571	6.9034359	.003039514
330	108900	35937000	18.1659021	6.9104232	.003030303
331	109561	36264691	18.1934054	6.9173964	.003021148
332	110224	36594368	18.2208672	6.9243556	.003012048
333	110889	36926037	18.2482876	6.9313008	.003003003
334	111556	37259704	18.2756669	6.9382321	.002994012
335	112225	37595375	18.3030052	6.9451496	.002985075
336	112896	37933056	18.3303028	6.9520533	.002976190
337	113569	38272753	18.3575598	6.9589434	.002967359
338	114244	38614472	18.3847763	6.9658198	.002958580
339	114921	38958219	18.4119526	6.9726826	.002949853
340	115600	39304000	18.4390889	6.9795321	.002941176
341	116281	39651821	18.4661853	6.9863681	.002932551
342	116964	40001688	18.4932420	6.9931906	.002923977
343	117649	40353607	18.5202592	7.0000000	.002915452
344	118336	40707584	18.5472370	7.0067962	.002906977
345	119025	41063625	18.5741756	7.0135791	.002898551
346	119716	41421736	18.6010752	7.0203490	.002890173
347	120409	41781923	18.6279360	7.0271058	.002881844
348	121104	42144192	18.6547581	7.0338497	.002873563
349	121801	42508549	18.6815417	7.0405806	.002865330
350	122500	42875000	18.7082869	7.0472987	.002857143
351	123201	43243551	18.7349940	7.0540041	.002849003
352	123904	43614208	18.7616630	7.0606967	.002840909
353	124609	43986977	18.7882942	7.0673767	.002832861
354	125316	44361864	18.8148877	7.0740440	.002824859
355	126025	44738875	18.8414437	7.0806988	.002816901
356	126736	45118016	18.8679623	7.0873411	.002808989
357	127449	45499293	18.8944436	7.0939709	.002801120
358	128164	45882712	18.9208879	7.1005885	.002793296
359	128881	46268279	18.9472953	7.1071937	.002785515
360	129600	46656000	18.9736660	7.1137866	.002777778
361	130321	47045881	19.0000000	7.1203674	.002770083
362	131044	47437928	19.0262976	7.1269360	.002762431
363	131769	47832147	19.0525589	7.1334925	.002754821
364	132496	48228544	19.0787840	7.1400370	.002747253
365	133225	48627125	19.1049732	7.1465695	.002739726
366	133956	49027896	19.1311265	7.1530901	.002732240
367	134689	[illegible]	19.1572441	7.1595988	.002724796
368	135424	[illegible]	[illegible]	7.1660957	.002717391
369	136161	50243409	19.2093727	7.1725809	.002710027
370	136900	50653000	19.2353841	7.1790544	.002702703
[illegible]	137641	51064811	19.2613603	7.1855162	.002695418
[illegible]	138384	51478848	19.2873015	7.1919663	.002688172

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
373	139129	51895117	19.3132079	7.1984050	.002680965
374	139876	52313624	19.3390796	7.2048322	.002673797
375	140625	52734375	19.3649167	7.2112479	.002666667
376	141376	53157376	19.3907194	7.2176522	.002659574
377	142129	53582633	19.4164878	7.2240450	.002652520
378	142884	54010152	19.4422221	7.2304268	.002645503
379	143641	54439939	19.4679223	7.2367972	.002638522
380	144400	54872000	19.4935887	7.2431565	.002631579
381	145161	55306341	19.5192213	7.2495045	.002624672
382	145924	55742968	19.5448203	7.2558415	.002617801
383	146689	56181887	19.5703858	7.2621675	.002610966
384	147456	56623104	19.5959179	7.2684824	.002604167
385	148225	57066625	19.6214169	7.2747864	.002597403
386	148996	57512456	19.6468827	7.2810794	.002590674
387	149769	57960603	19.6723156	7.2873617	.002583979
388	150544	58411072	19.6977156	7.2936330	.002577320
389	151321	58863869	19.7230829	7.2998936	.002570694
390	152100	59319000	19.7484177	7.3061436	.002564103
391	152881	59776471	19.7737199	7.3123828	.002557545
392	153664	60236288	19.7989899	7.3186114	.002551020
393	154449	60698457	19.8242276	7.3248295	.002544529
394	155236	61162984	19.8494332	7.3310369	.002538071
395	156025	61629875	19.8746069	7.3372339	.002531646
396	156816	62099136	19.8997487	7.3434205	.002525253
397	157609	62570773	19.9248588	7.3495966	.002518892
398	158404	63044792	19.9499373	7.3557624	.002512563
399	159201	63521199	19.9749844	7.3619178	.002506266
400	160000	64000000	20.0000000	7.3680630	.002500000
401	160801	64481201	20.0249844	7.3741979	.002493766
402	161604	64964808	20.0499377	7.3803227	.002487562
403	162409	65450827	20.0748599	7.3864373	.002481390
404	163216	65939264	20.0997512	7.3925418	.002475248
405	164025	66430125	20.1246118	7.3986363	.002469136
406	164836	66923416	20.1494417	7.4047206	.002463054
407	165649	67419143	20.1742410	7.4107950	.002457002
408	166464	67917312	20.1990099	7.4168595	.002450980
409	167281	68417929	20.2237484	7.4229142	.002444988
410	168100	68921000	20.2484567	7.4289589	.002439024
411	168921	69426531	20.2731349	7.4349938	.002433090
412	169744	69934528	20.2977831	7.4410189	.002427184
413	170569	70444997	20.3224014	7.4470342	.002421308
414	171396	70957944	20.3469899	7.4530399	.002415459
415	172225	71473375	20.3715488	7.4590359	.002409639
416	173056	71991296	20.3960781	7.4650223	.002403846
417	173889	72511713	20.4205779	7.4709991	.002398082
418	174724	73034632	20.4450483	7.4769664	.002392344
419	175561	73560059	20.4694895	7.4829242	.002386635
420	176400	74088000	20.4939015	7.4888724	.002380952
421	177241	74618461	20.5182845	7.4948113	.002375297
422	178084	75151448	20.5426386	7.5007406	.002369668
423	178929	75686967	20.5669638	7.5066607	.002364066
424	179776	76225024	20.5912603	7.5125715	.002358491
425	180625	76765625	20.6155281	7.5184730	.002352941
426	181476	77308776	20.6397674	7.5243652	.002347418
427	182329	77854483	20.6639783	7.5302482	.002341920
428	183184	78402752	20.6881609	7.5361221	.002336449
429	184041	78953589	20.7123152	7.5419867	.002331002
430	184900	79507000	20.7364414	7.5478423	.00232558[illegible]
431	185761	80062991	20.7605395	7.5536888	.00232018[illegible]
432	186624	80621568	20.7846097	7.5595263	.002314[illegible]
433	187489	81182737	20.8086520	7.5653548	.00230[illegible]
434	188356	81746504	20.8326667	7.5711743	.00[illegible]

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
435	189225	82312875	20.8566536	7.5769849	.002298851
436	190096	82881856	20.8806130	7.5827865	.002293578
437	190969	83453453	20.9045450	7.5885793	.002288330
438	191844	84027672	20.9284495	7.5943633	.002283105
439	192721	84604519	20.9523268	7.6001385	.002277904
440	193600	85184000	20.9761770	7.6059049	.002272727
441	194481	85766121	21.0000000	7.6116626	.002267574
442	195364	86350888	21.0237960	7.6174116	.002262443
443	196249	86938307	21.0475652	7.6231519	.002257336
444	197136	87528384	21.0713075	7.6288837	.002252252
445	198025	88121125	21.0950231	7.6346067	.002247191
446	198916	88716536	21.1187121	7.6403213	.002242152
447	199809	89314623	21.1423745	7.6460272	.002237136
448	200704	89915392	21.1660105	7.6517247	.002232143
449	201601	90518849	21.1896201	7.6574138	.002227171
450	202500	91125000	21.2132034	7.6630943	.002222222
451	203401	91733851	21.2367606	7.6687665	.002217295
452	204304	92345408	21.2602916	7.6744303	.002212389
453	205209	92959677	21.2837967	7.6800857	.002207506
454	206116	93576664	21.3072758	7.6857328	.002202643
455	207025	94196375	21.3307290	7.6913717	.002197802
456	207936	94818816	21.3541565	7.6970023	.002192982
457	208849	95443993	21.3775583	7.7026246	.002188184
458	209764	96071912	21.4009346	7.7082388	.002183406
459	210681	96702579	21.4242853	7.7138448	.002178649
460	211600	97336000	21.4476106	7.7194426	.002173913
461	212521	97972181	21.4709106	7.7250325	.002169197
462	213444	98611128	21.4941853	7.7306141	.002164502
463	214369	99252847	21.5174348	7.7361877	.002159827
464	215296	99897344	21.5406592	7.7417532	.002155172
465	216225	100544625	21.5638587	7.7473109	.002150538
466	217156	101194696	21.5870331	7.7528606	.002145923
467	218089	101847563	21.6101828	7.7584023	.002141328
468	219024	102503232	21.6333077	7.7639361	.002136752
469	219961	103161709	21.6564078	7.7694620	.002132196
470	220900	103823000	21.6794834	7.7749801	.002127660
471	221841	104487111	21.7025344	7.7804904	.002123142
472	222784	105154048	21.7255610	7.7859928	.002118644
473	223729	105823817	21.7485632	7.7914875	.002114165
474	224676	106496424	21.7715411	7.7969745	.002109705
475	225625	107171875	21.7944947	7.8024538	.002105263
476	226576	107850176	21.8174242	7.8079254	.002100840
477	227529	108531333	21.8403297	7.8133892	.002096436
478	228484	109215352	21 8632111	7.8188456	.002092050
479	229441	109902239	21.8860686	7.8242942	.002087683
480	230400	110592000	21.9089023	7.8297353	.002083333
481	231361	111284641	21.9317122	7.8351688	.002079002
482	232324	111980168	21.9544984	7 8405949	.002074689
483	233289	112678587	21.9772610	7.8460134	.002070393
484	234256	113379904	22.0000000	7.8514244	.002066116
485	235225	114084125	22.0227155	7.8568281	.002061856
486	236196	114791256	22 0454077	7.8622242	.002057613
487	237169	115501303	22.0680765	7.8676130	.002053388
488	238144	116214272	22.0907220	7.8729944	.002049180
489	239121	116930169	22.1133444	7.8783684	.002044990
490	240100	117649000	22.1359436	7.8837352	.002040816
491	241081	118370771	22.1585198	7.8890946	.002036660
492	242064	119095488	22.1810730	7.8944468	.002032520
493	243049	119823157	22.2036033	7.8997917	.002028398
'94	244036	120553784	22.2261108	7.9051294	.002024291
95	245025	121287375	22.2485955	7.9104599	.002020202
[illegible]	246016	122023936	22.2710575	7.9157832	.002016129

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
497	247009	122763473	22.2934968	7.9210994	.002012072
498	248004	123505992	22.3159136	7.9264085	.002008032
499	249001	124251499	22.3383079	7.9317104	.002004008
500	250000	125000000	22.3606798	7.9370053	.002000000
501	251001	125751501	22.3830293	7.9422931	.001996008
502	252004	126506008	22.4053565	7.9475739	.001992032
503	253009	127263527	22.4276615	7.9528477	.001988072
504	254016	128024064	22.4499443	7.9581144	.001984127
505	255025	128787625	22.4722051	7.9633743	.001980198
506	256036	129554216	22.4944438	7.9686271	.001976285
507	257049	130323843	22.5166605	7.9738731	.001972387
508	258064	131096512	22.5388553	7.9791122	.001968504
509	259081	131872229	22.5610283	7.9843444	.001964637
510	260100	132651000	22.5831796	7.9895697	.001960784
511	261121	133432831	22.6053091	7.9947883	.001956947
512	262144	134217728	22.6274170	8.0000000	.001953125
513	263169	135005697	22.6495033	8.0052049	.001949318
514	264196	135796744	22.6715681	8.0104032	.001945525
515	265225	136590875	22.6936114	8.0155946	.001941748
516	266256	137388096	22.7156334	8.0207794	.001937984
517	267289	138188413	22.7376340	8.0259574	.001934236
518	268324	138991832	22.7596134	8.0311287	.001930502
519	269361	139798359	22.7815715	8.0362935	.001926782
520	270400	140608000	22.8035085	8.0414515	.001923077
521	271441	141420761	22.8254244	8.0466030	.001919386
522	272484	142236648	22.8473193	8.0517479	.001915709
523	273529	143055667	22.8691933	8.0568862	.001912046
524	274576	143877824	22.8910463	8.0620180	.001908397
525	275625	144703125	22.9128785	8.0671432	.001904762
526	276676	145531576	22.9346899	8.0722620	.001901141
527	277729	146363183	22.9564806	8.0773743	.001897533
528	278784	147197952	22.9782506	8.0824800	.001893939
529	279841	148035889	23.0000000	8.0875794	.001890359
530	280900	148877000	23.0217289	8.0926723	.001886792
531	281961	149721291	23.0434372	8.0977589	.001883239
532	283024	150568768	23.0651252	8.1028390	.001879699
533	284089	151419437	23.0867928	8.1079128	.001876173
534	285156	152273304	23.1084400	8.1129803	.001872659
535	286225	153130375	23.1300670	8.1180414	.001869159
536	287296	153990656	23.1516738	8.1230962	.001865672
537	288369	154854153	23.1732605	8.1281447	.001862197
538	289444	155720872	23.1948270	8.1331870	.001858735
539	290521	156590819	23.2163735	8.1382230	.001855288
540	291600	157464000	23.2379001	8.1432529	.001851853
541	292681	158340421	23.2594067	8.1482765	.001848429
542	293764	159220088	23.2808935	8.1532939	.001845018
543	294849	160103007	23.3023604	8.1583051	.001841621
544	295936	160989184	23.3238076	8.1633102	.001838235
545	297025	161878625	23.3452351	8.1683092	.001834862
546	298116	162771336	23.3666429	8.1733020	.001831503
547	299209	163667323	23.3880311	8.1782888	.001828154
548	300304	164566592	23.4093998	8.1832695	.001824818
549	301401	165469149	23.4307490	8.1882441	.001821494
550	302500	166375000	23.4520788	8.1932127	.001818182
551	303601	167284151	23.4733892	8.1981753	.001814882
552	304704	168196608	23.4946802	8.2031319	.001811594
553	305809	169112377	23.5159520	8.2080825	.001808318
554	306916	170031464	23.5372046	8.2130271	.001805054
555	308025	170953875	23.5584380	8.2179657	.001801802
556	309136	171879616	23.5796522	8.2228985	.001798561
557	310249	172808693	23.6008474	8.2278254	.001795332
558	311364	173741112	23.6220236	8.2327463	.001792115

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
559	312481	174676879	23 6431808	8.2376614	.001788909
560	313600	175616000	23 6643191	8.2425706	.001785714
561	314721	176558481	23.6854386	8.2474740	.001782531
562	315844	177504328	23.7065392	8.2523715	.001779359
563	316969	178453547	23.7276210	8.2572633	.001776199
564	318096	179406144	23.7486842	8.2621492	.001773050
565	319225	180362125	23.7697286	8.2670294	.001769912
566	320356	181321496	23.7907545	8.2719039	.001766784
567	321489	182284263	23.8117618	8.2767726	.001763668
568	322624	183250432	23.8327506	8.2816355	.001760563
569	323761	184220000	23.8537209	8.2864928	.001757469
570	324900	185193000	23.8746728	8.2913444	.001754386
571	326041	186169411	23.8956063	8.2961903	.001751313
572	327184	187149248	23.9165215	8.3010304	.001748252
573	328329	188132517	23.9374184	8.3058651	.001745201
574	329476	189119224	23.9582971	8.3106941	.001742160
575	330625	190109375	23.9791576	8.3155175	.001739130
576	331776	191102976	24.0000000	8.3203353	.001736111
577	332929	192100033	24.0208243	8.3251475	.001733102
578	334084	193100552	24.0416306	8.3299542	.001730104
579	335241	194104539	24.0624188	8.3347553	.001727116
580	336400	195112000	24.0831891	8.3395509	.001724138
581	337561	196122941	24.1039416	8.3443410	.001721170
582	338724	197137368	24.1246762	8.3491256	.001718213
583	339889	198155287	24.1453929	8.3539047	.001715266
584	341056	199176704	24.1660919	8.3586784	.001712329
585	342225	200201625	24.1867732	8.3634466	.001709402
586	343396	201230056	24.2074369	8.3682095	.001706485
587	344569	202262003	24.2280829	8.3729668	.001703578
588	345744	203297472	24.2487113	8.3777188	.001700680
589	346921	204336469	24.2693222	8.3824653	.001697793
590	348100	205379000	24.2899156	8.3872065	.001694915
591	349281	206425071	24.3104916	8.3919423	.001692047
592	350464	207474688	24.3310501	8.3966729	.001689189
593	351649	208527857	24.3515913	8.4013981	.001686341
594	352836	209584584	24.3721152	8.4061180	.001683502
595	354025	210644875	24.3926218	8.4108326	.001680672
596	355216	211708736	24.4131112	8.4155419	.001677852
597	356409	212776173	24.4335834	8.4202460	.001675042
598	357604	213847192	24.4540385	8.4249448	.001672241
599	358801	214921799	24.4744765	8.4296383	.001669449
600	360000	216000000	24.4948974	8.4343267	.001666667
601	361201	217081801	24.5153013	8.4390098	.001663894
602	362404	218167208	24.5356883	8.4436877	.001661130
603	363609	219256227	24.5560583	8.4483605	.001658375
604	364816	220348864	24.5764115	8.4530281	.001655629
605	366025	221445125	24.5967478	8.4576906	.001652893
606	367236	222545016	24.6170673	8.4623479	.001650165
607	368449	223648543	24.6373700	8.4670001	.001647446
608	369664	224755712	24.6576560	8.4716471	.001644737
609	370881	225866529	24.6779254	8.4762892	.001642036
610	372100	226981000	24.6981781	8.4809261	.001639344
611	373321	228099131	24.7184142	8.4855579	.001636661
612	374544	229220928	24.7386338	8.4901848	.001633987
613	375769	230346397	24.7588368	8.4948065	.001631321
614	376996	231475544	24.7790234	8.4994233	.001628664
615	378225	232608375	24.7991935	8.5040350	.001626016
616	379456	233744896	24.8193473	8.5086417	.001623377
617	380689	234885113	24.8394847	8.5132435	.001620746
618	381924	236029032	24.8596058	8.5178403	.001618123
619	383161	237176659	24.8797106	8.5224321	.001615509
620	384400	238328000	24.8997992	8.5270189	.001612903

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
745	555025	413493625	27.2946881	9.0653677	.001342282
746	556516	415160936	27.3130006	9.0694220	.001340483
747	558009	416832723	27.3313007	9.0734726	.001338688
748	559504	418508992	27.3495887	9.0775197	.001336898
749	561001	420189749	27.3678644	9.0815631	.001335113
750	562500	421875000	27.3861279	9.0856030	.001333333
751	564001	423564751	27.4043792	9.0896392	.001331558
752	565504	425259008	27.4226184	9.0936719	.001329787
753	567009	426957777	27.4408455	9.0977010	.001328021
754	568516	428661064	27.4590604	9.1017265	.001326260
755	570025	430368875	27.4772633	9.1057485	.001324503
756	571536	432081216	27.4954542	9.1097669	.001322751
757	573049	433798093	27.5136330	9.1137818	.001321004
758	574564	435519512	27.5317998	9.1177931	.001319261
759	576081	437245479	27.5499546	9.1218010	.001317523
760	577600	438976000	27.5680975	9.1258053	.001315789
761	579121	440711081	27.5862284	9.1298061	.001314060
762	580644	442450728	27.6043475	9.1338034	.001312336
763	582169	444194947	27.6224546	9.1377971	.001310616
764	583696	445943744	27.6405499	9.1417874	.001308901
765	585225	447697125	27.6586334	9.1457742	.001307190
766	586756	449455096	27.6767050	9.1497576	.001305483
767	588289	451217663	27.6947648	9.1537375	.001303781
768	589824	452984832	27.7128129	9.1577139	.001302083
769	591361	454756609	27.7308492	9.1616869	.001300390
770	592900	456533000	27.7488739	9.1656565	.001298701
771	594441	458314011	27.7668868	9.1696225	.001297017
772	595984	460099648	27.7848880	9.1735852	.001295337
773	597529	461889917	27.8028775	9.1775445	.001293661
774	599076	463684824	27.8208555	9.1815003	.001291990
775	600625	465484375	27.8388218	9.1854527	.001290323
776	602176	467288576	27.8567766	9.1894018	.001288660
777	603729	469097433	27.8747197	9.1933474	.001287001
778	605284	470910952	27.8926514	9.1972897	.001285347
779	606841	472729139	27.9105715	9.2012286	.001283697
780	608400	474552000	27.9284801	9.2051641	.001282051
781	609961	476379541	27.9463772	9.2090962	.001280410
782	611524	478211768	27.9642629	9.2130250	.001278772
783	613089	480048687	27.9821372	9.2169505	.001277139
784	614656	481890304	28.0000000	9.2208726	.001275510
785	616225	483736625	28.0178515	9.2247914	.001273885
786	617796	485587656	28.0356915	9.2287068	.001272265
787	619369	487443403	28.0535203	9.2326189	.001270648
788	620944	489303872	28.0713377	9.2365277	.001269036
789	622521	491169069	28.0891438	9.2404333	.001267427
790	624100	493039000	28.1069386	9.2443355	.001265823
791	625681	494913671	28.1247222	9.2482344	.001264223
792	627264	496793088	28.1424946	9.2521300	.001262626
793	628849	498677257	28.1602557	9.2560224	.001261034
794	630436	500566184	28.1780056	9.2599114	.001259446
795	632025	502459875	28.1957444	9.2637973	.001257862
796	633616	504358336	28.2134720	9.2676798	.001256281
797	635209	506261573	28.2311884	9.2715592	.001254705
798	636804	508169592	28.2488938	9.2754352	.001253133
799	638401	510082399	28.2665881	9.2793081	.001251564
800	640000	512000000	28.2842712	9.2831777	.001250000
801	641601	513922401	28.3019434	9.2870440	.001248439
802	643204	515849608	28.3196045	9.2909072	.001246883
	644809	517781627	28.3372546	9.2947671	.001245330
	646416	519718464	28.3548938	9.2986239	.001243781
	648025	521660125	28.3725219	9.3024775	.001242236
	649636	523606616	28.3901391	9.3063278	.001240695

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
807	651249	525557943	28.4077454	9.3101750	.001239157
808	652864	527514112	28.4253408	9.3140190	.001237624
809	654481	529475129	28.4429253	9.3178599	.001236094
810	656100	531441000	28.4604989	9.3216975	.001234568
811	657721	533411731	28.4780617	9.3255320	.001233046
812	659344	535387328	28.4956137	9.3293634	.001231527
813	660969	537367797	28.5131549	9.3331916	.001230012
814	662596	539353144	28.5306852	9.3370167	.001228501
815	664225	541343375	28.5482048	9.3408386	.001226994
816	665856	543338496	28.5657137	9.3446575	.001225490
817	667489	545338513	28.5832119	9.3484731	.001223990
818	669124	547343432	28.6006993	9.3522857	.001222494
819	670761	549353259	28.6181760	9.3560952	.001221001
820	672400	551368000	28.6356421	9.3599016	.001219512
821	674041	553387661	28.6530976	9.3637049	.001218027
822	675684	555412248	28.6705424	9.3675051	.001216545
823	677329	557441767	28.6879766	9.3713022	.001215067
824	678976	559476224	28.7054002	9.3750963	.001213592
825	680625	561515625	28.7228132	9.3788873	.001212121
826	682276	563559976	28.7402157	9.3826752	.001210654
827	683929	565609283	28.7576077	9.3864600	.001209190
828	685584	567663552	28.7749891	9.3902419	.001207729
829	687241	569722789	28.7923601	9.3940206	.001206273
830	688900	571787000	28.8097206	9.3977964	.001204819
831	690561	573856191	28.8270706	9.4015691	.001203369
832	692224	575930368	28.8444102	9.4053387	.001201923
833	693889	578009537	28.8617394	9.4091054	.001200480
834	695556	580093704	28.8790582	9.4128690	.001199041
835	697225	582182875	28.8963666	9.4166297	.001197605
836	698896	584277056	28.9136646	9.4203873	.001196172
837	700569	586376253	28.9309523	9.4241420	.001194743
838	702244	588480472	28.9482297	9.4278936	.001193317
839	703921	590589719	28.9654967	9.4316423	.001191895
840	705600	592704000	28.9827535	9.4353880	.001190476
841	707281	594823321	29.0000000	9.4391307	.001189061
842	708964	596947688	29.0172363	9.4428704	.001187648
843	710649	599077107	29.0344623	9.4466072	.001186240
844	712336	601211584	29.0516781	9.4503410	.001184834
845	714025	603351125	29.0688837	9.4540719	.001183432
846	715716	605495736	29.0860791	9.4577999	.001182033
847	717409	607645423	29.1032644	9.4615249	.001180638
848	719104	609800192	29.1204396	9.4652470	.001179245
849	720801	611960049	29.1376046	9.4689661	.001177856
850	722500	614125000	29.1547595	9.4726824	.001176471
851	724201	616295051	29.1719043	9.4763957	.001175088
852	725904	618470208	29.1890390	9.4801061	.001173709
853	727609	620650477	29.2061637	9.4838136	.001172333
854	729316	622835864	29.2232784	9.4875182	.001170960
855	731025	625026375	29.2403830	9.4912200	.001169591
856	732736	627222016	29.2574777	9.4949188	.001168224
857	734449	629422793	29.2745623	9.4986147	.001166861
858	736164	631628712	29.2916370	9.5023078	.001165501
859	737881	633839779	29.3087018	9.5059980	.001164144
860	739600	636056000	29.3257566	9.5096854	.001162791
861	741321	638277381	29.3428015	9.5133699	.001161440
862	743044	640503928	29.3598365	9.5170515	.001160093
863	744769	642735647	29.3768616	9.5207303	.001158749
864	746496	644972544	29.3938769	9.5244063	.001157407
865	748225	647214625	29.4108823	9.5280794	.001156069
866	749956	649461896	29.4278779	9.5317497	.001154734
867	751689	651714363	29.4448637	9.5354172	.001153403
868	753424	653972032	29.4618397	9.5390818	.001152074

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
869	755161	656234909	29.4788059	9.5427437	.001150748
870	756900	658503000	29.4957624	9.5464027	.001149425
871	758641	660776311	29.5127091	9.5500589	.001148106
872	760384	663054848	29.5296461	9.5537123	.001146789
873	762129	665338617	29.5465734	9.5573630	.001145475
874	763876	667627624	29.5634910	9.5610108	.001144165
875	765625	669921875	29.5803989	9.5646559	.001142857
876	767376	672221376	29.5972972	9.5682982	.001141553
877	769129	674526133	29.6141858	9.5719377	.001140251
878	770884	676836152	29.6310648	9.5755745	.001138952
879	772641	679151439	29.6479342	9.5792085	.001137656
880	774400	681472000	29.6647939	9.5828397	.001136364
881	776161	683797841	29.6816442	9.5864682	.001135074
882	777924	686128968	29.6984848	9.5900939	.001133787
883	779689	688465387	29.7153159	9.5937169	.001132503
884	781456	690807104	29.7321375	9.5973373	.001131222
885	783225	693154125	29.7489496	9.6009548	.001129944
886	784996	695506456	29.7657521	9.6045696	.001128668
887	786769	697864103	29.7825452	9.6081817	.001127396
888	788544	700227072	29.7993289	9.6117911	.001126126
889	790321	702595369	29.8161030	9.6153977	.001124859
890	792100	704969000	29.8328678	9.6190017	.001123596
891	793881	707347971	29.8496231	9.6226030	.001122334
892	795664	709732288	29.8663690	9.6262016	.001121076
893	797449	712121957	29.8831056	9.6297975	.001119821
894	799236	714516984	29.8998328	9.6333907	.001118568
895	801025	716917375	29.9165506	9.6369812	.001117318
896	802816	719323136	29.9332591	9.6405690	.001116071
897	804609	721734273	29.9499583	9.6441542	.001114827
898	806404	724150792	29.9666481	9.6477367	.001113586
899	808201	726572699	29.9833287	9.6513166	.001112347
900	810000	729000000	30.0000000	9.6548938	.001111111
901	811801	731432701	30.0166620	9.6584684	.001109878
902	813604	733870808	30.0333148	9.6620403	.001108647
903	815409	736314327	30.0499584	9.6656096	.001107420
904	817216	738763264	30.0665928	9.6691762	.001106195
905	819025	741217625	30.0832179	9.6727403	.001104972
906	820836	743677416	30.0998339	9.6763017	.001103753
907	822649	746142643	30.1164407	9.6798604	.001102536
908	824464	748613312	30.1330383	9.6834166	.001101322
909	826281	751089429	30.1496269	9.6869701	.001100110
910	828100	753571000	30.1662063	9.6905211	.001098901
911	829921	756058031	30.1827765	9.6940694	.001097695
912	831744	758550528	30.1993377	9.6976151	.001096491
913	833569	761048497	30.2158899	9.7011583	.001095290
914	835396	763551944	30.2324329	9.7046989	.001094092
915	837225	766060875	30.2489669	9.7082369	.001092896
916	839056	768575296	30.2654919	9.7117723	.001091703
917	840889	771095213	30.2820079	9.7153051	.001090513
918	842724	773620632	30.2985148	9.7188354	.001089325
919	844561	776151559	30.3150128	9.7223631	.001088139
920	846400	778688000	30.3315018	9.7258883	.001086957
921	848241	781229961	30.3479818	9.7294109	.001085776
922	850084	783777448	30.3644529	9.7329309	.001084599
923	851929	786330467	30.3809151	9.7364484	.001083423
924	853776	788889024	30.3973683	9.7399634	.001082251
925	855625	791453125	30.4138127	9.7434758	.001081081
926	857476	794022776	30.4302481	9.7469857	.001079914
927	859329	796597983	30.4466747	9.7504930	.001078749
928	861184	799178752	30.4630924	9.7539979	.001077586
929	863041	801765089	30.4795013	9.7575002	.001076426
930	864900	804357000	30.4959014	9.7610001	001075269

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
931	866761	806954491	30.5122926	9.7644974	.001074114
932	868624	809557568	30.5286750	9.7679922	.001072961
933	870489	812166237	30.5450487	9.7714845	.001071811
934	872356	814780504	30.5614136	9.7749743	.001070664
935	874225	817400375	30.5777697	9.7784616	.001069519
936	876096	820025856	30.5941171	9.7819466	.001068376
937	877969	822656953	30.6104557	9.7854288	.001067236
938	879844	825293672	30.6267857	9.7889087	.001066098
939	881721	827936019	30.6431069	9.7923861	.001064963
940	883600	830584000	30.6594194	9.7958611	.001063830
941	885481	833237621	30.6757233	9.7993336	.001062699
942	887364	835896888	30.6920185	9.8028036	.001061571
943	889249	838561807	30.7083051	9.8062711	.001060445
944	891136	841232384	30.7245830	9.8097362	.001059322
945	893025	843908625	30.7408523	9.8131989	.001058201
946	894916	846590536	30.7571130	9.8166591	.001057082
947	896809	849278123	30.7733651	9.8201169	.001055966
948	898704	851971392	30.7896086	9.8235723	.001054852
949	900601	854670349	30.8058436	9.8270252	.001053741
950	902500	857375000	30.8220700	9.8304757	.001052632
951	904401	860085351	30.8382879	9.8339238	.001051525
952	906304	862801408	30.8544972	9.8373695	.001050420
953	908209	865523177	30.8706981	9.8408127	.001049318
954	910116	868250664	30.8868904	9.8442536	.001048218
955	912025	870983875	30.9030743	9.8476920	.001047120
956	913936	873722816	30.9192497	9.8511280	.001046025
957	915849	876467493	30.9354166	9.8545617	.001044932
958	917764	879217912	30.9515751	9.8579929	.001043841
959	919681	881974079	30.9677251	9.8614218	.001042753
960	921600	884736000	30.9838668	9.8648483	.001041667
961	923521	887503681	31.0000000	9.8682724	.001040583
962	925444	890277128	31.0161248	9.8716941	.001039501
963	927369	893056347	31.0322413	9.8751135	.001038422
964	929296	895841344	31.0483494	9.8785305	.001037344
965	931225	898632125	31.0644491	9.8819451	.001036269
966	933156	901428696	31.0805405	9.8853574	.001035197
967	935089	904231063	31.0966236	9.8887673	.001034126
968	937024	907039232	31.1126984	9.8921749	.001033058
969	938961	909853209	31.1287648	9.8955801	.001031992
970	940900	912673000	31.1448230	9.8989830	.001030928
971	942841	915498611	31.1608729	9.9023835	.001029866
972	944784	918330048	31.1769145	9.9057817	.001028807
973	946729	921167317	31.1929479	9.9091776	.001027749
974	948676	924010424	31.2089731	9.9125712	.001026694
975	950625	926859375	31.2249900	9.9159624	.001025641
976	952576	929714176	31.2409987	9.9193513	.001024590
977	954529	932574833	31.2569992	9.9227379	.001023541
978	956484	935441352	31.2729915	9.9261222	.001022495
979	958441	938313739	31.2889757	9.9295042	.001021450
980	960400	941192000	31.3049517	9.9328839	.001020408
981	962361	944076141	31.3209195	9.9362613	.001019368
982	964324	946966168	31.3368792	9.9396363	.001018330
983	966289	949862087	31.3528308	9.9430092	.001017294
984	968256	952763904	31.3687743	9.9463797	.001016260
985	970225	955671625	31.3847097	9.9497479	.001015228
986	972196	958585256	31.4006369	9.9531138	.001014199
987	974169	961504803	31.4165561	9.9564775	.001013171
988	976144	964430272	31.4324673	9.9598389	.001012146
989	978121	967361669	31.4483704	9.9631981	.001011122
990	980100	970299000	31.4642654	9.9665549	.001010101
991	982081	973242271	31.4801525	9.9699095	.001009082
992	984064	976191488	31.4960315	9.9732619	.001008065

No.	Squares.	Cubes.	Square Roots.	Cube Roots.	Reciprocals.
993	986049	979146657	31.5119025	9.9766120	.001007049
994	988036	982107784	31.5277655	9.9799599	.001006036
995	990025	985074875	31.5436206	9.9833055	.001005025
996	992016	988047936	31.5594677	9.9866488	.001004016
997	994009	991026973	31.5753068	9.9899900	.001003009
998	996004	994011992	31.5911380	9.9933289	.001002004
999	998001	997002999	31.6069613	9.9966656	.001001001
1000	1000000	1000000000	31.6227766	10.0000000	.001000000
1001	1002001	1003003001	31.6385840	10.0033322	.0009990010
1002	1004004	1006012008	31.6543836	10.0066622	.0009980040
1003	1006009	1009027027	31.6701752	10.0099899	.0009970090
1004	1008016	1012048064	31.6859590	10.0133155	.0009960159
1005	1010025	1015075125	31.7017349	10.0166389	.0009950249
1006	1012036	1018108216	31.7175030	10.0199601	.0009940358
1007	1014049	1021147343	31.7332633	10.0232791	.0009930487
1008	1016064	1024192512	31.7490157	10.0265958	.0009920635
1009	1018081	1027243729	31.7647603	10.0299104	.0009910803
1010	1020100	1030301000	31.7804972	10.0332228	.0009900990
1011	1022121	1033364331	31.7962232	10.0365330	.0009891197
1012	1024144	1036433728	31.8119474	10.0398410	.0009881423
1013	1026169	1039509197	31.8276609	10.0431469	.0009871668
1014	1028196	1042590744	31.8433666	10.0464506	.0009861933
1015	1030225	1045678375	31.8590646	10.0497521	.0009852217
1016	1032256	1048772096	31.8747549	10.0530514	.0009842520
1017	1034289	1051871913	31.8904374	10.0563485	.0009832842
1018	1036324	1054977832	31.9061123	10.0596435	.0009823183
1019	1038361	1058089859	31.9217794	10.0629364	.0009813543
1020	1040400	1061208000	31.9374388	10.0662271	.0009803922
1021	1042441	1064332261	31.9530906	10.0695156	.0009794319
1022	1044484	1067462648	31.9687347	10.0728020	.0009784736
1023	1046529	1070599167	31.9843712	10.0760863	.0009775171
1024	1048576	1073741824	32.0000000	10.0793684	.0009765625
1025	1050625	1076890625	32.0156212	10.0826484	.0009756098
1026	1052676	1080045576	32.0312348	10.0859262	.0009746589
1027	1054729	1083206683	32.0468407	10.0892019	.0009737098
1028	1056784	1086373952	32.0624391	10.0924755	.0009727626
1029	1058841	1089547389	32.0780298	10.0957469	.0009718173
1030	1060900	1092727000	32.0936131	10.0990163	.0009708738
1031	1062961	1095912791	32.1091887	10.1022835	.0009699321
1032	1065024	1099104768	32.1247568	10.1055487	.0009689922
1033	1067089	1102302937	32.1403173	10.1088117	.0009680542
1034	1069156	1105507304	32.1558704	10.1120726	.0009671180
1035	1071225	1108717875	32.1714159	10.1153314	.0009661836
1036	1073296	1111934656	32.1869539	10.1185882	.0009652510
1037	1075369	1115157653	32.2024844	10.1218428	.0009643202
1038	1077444	1118386872	32.2180074	10.1250953	.0009633911
1039	1079521	1121622319	32.2335229	10.1283457	.0009624639
1040	1081600	1124864000	32.2490310	10.1315941	.0009615385
1041	1083681	1128111921	32.2645316	10.1348403	.0009606148
1042	1085764	1131366088	32.2800248	10.1380845	.0009596929
1043	1087849	1134626507	32.2955105	10.1413266	.0009587738
1044	1089936	1137893184	32.3109888	10.1445667	.0009578544
1045	1092025	1141166125	32.3264598	10.1478047	.0009569378
1046	1094116	1144445336	32.3419233	10.1510406	.0009560229
1047	1096209	1147730823	32.3573794	10.1542744	.0009551098
1048	1098304	1151022592	32.3728281	10.1575062	.0009541985
1049	1100401	1154320649	32.3882695	10.1607359	.0009532888
1050	1102500	1157625000	32.4037035	10.1639636	.0009523810
1051	1104601	1160935651	32.4191301	10.1671893	.0009514748
1052	1106704	1164252608	32.4345495	10.1704129	.0009505703
1053	1108809	1167575877	32.4499615	10.1736344	.0009496676
1054	1110916	1170905464	32.4653662	10.1768539	.0009487666

WEIGHTS AND MEASURES.

Measures of Length.

12 inches = 1 foot.
3 feet = 1 yard = 36 inches.
5½ yards = 1 rod = 198 inches = 16½ ft.
40 rods = 1 furlong = 7920 inches = 660 ft. = 220 yds.
8 furlongs = 1 mile = 63360 inches = 5280 ft. = 1760 yds. [= 320 rods.
1 yard = 0.0005682 of a mile.

GUNTER'S CHAIN.

7.92 inches = 1 link.
100 links = 1 chain = 4 rods = 66 feet.
80 chains = 1 mile.

ROPES AND CABLES.

6 feet = 1 fathom. 120 fathoms = 1 cable's length.

Table showing Inches expressed in Decimals of a Foot.

GEOGRAPHICAL AND NAUTICAL.

1 degree of a great circle of the earth = 69.77 statute miles.
1 mile = 2046.58 yards.

SHOEMAKERS' MEASURE.

No. 1 is 4.125 inches in length, and every succeeding number is .333 of an inch.

There are 28 numbers or divisions, in two series of numbers, viz., from 1 to 13, and 1 to 15.

MISCELLANEOUS.

1 palm = 3 inches.	1 span = 9 inches.
1 hand = 4 inches.	1 meter = 3.2809 feet.

Measures of Surface.

144 square inches = 1 square foot.
9 square feet = 1 square yard = 1296 square inches.
100 square feet = 1 square (architects' measure).

LAND.

$30\frac{1}{4}$ square yards	= 1 square rod.	
40 square rods	= 1 square rood	= 1210 square yards.
4 square roods } 10 square chains }	= 1 acre = 160 square rods.	= 4840 square yards.
640 acres	= 1 square mile	= 3097600 square yards = 102400 sq. rods = 2560 sq. roods.
208.71 feet square	= 1 acre.	

A *section* of land is a square mile, and a *quarter-section* is 160 acres.

Measures of Volume.

1 gallon liquid measure = 231 cubic inches, and contains 8.339 avoirdupois pounds of distilled water at 39.8° F.

1 gallon dry measure = 268.8 cubic inches.

1 bushel (*Winchester*) contains 2150.42 cubic inches, or 77.627 pounds distilled water at 39.8° F.

A heaped bushel contains 2747.715 cubic inches.

DRY.

2 pints = 1 quart = 67.2 cubic inches.
4 quarts = 1 gallon = 8 pints = 268.8 cubic inches.
2 gallons = 1 peck = 16 pints = 8 quarts = 537.6 cubic inches.
4 pecks = 1 bushel = 64 pints = 32 quarts = 8 gals. = 2150.42 cu. in.
1 chaldron = 36 heaped bushels = 57.244 cubic feet.
1 cord of wood = 128 cubic feet.

LIQUID.

4 gills = 1 pint.
2 pints = 1 quart = 8 gills.
4 quarts = 1 gallon = 32 gills = 8 pints.

In the United States and Great Britain 1 barrel of wine or brandy = 31½ gallons, and contains 4.211 cubic feet.

A hogshead is 63 gallons, but this term is often applied to casks of various capacities.

Cubic Measure.

7,5 gal = 1728 cubic inches = 1 foot.
27 cubic feet = 1 yard.

In measuring wood, a pile of wood cut 4 feet long, piled 4 feet high, and 8 feet on the ground, making 128 cubic feet, is called a cord. Cord of limestone = 5 tons

16 cubic feet make one cord foot.

A perch of stone is 16½ feet long, 1 foot high, and 1½ feet thick, and contains 24¾ cubic feet.

A perch of stone is, however, often computed differently in different localities; thus, in Philadelphia, 22 cubic feet are called a perch, and in some of the New-England States a perch is computed at 16½ cubic feet.

A ton, in computing the tonnage of ships and other vessels, is 100 cubic feet of their internal space.

Fluid Measure.

60 minims = 1 fluid drachm.
8 fluid drachms = 1 ounce.
16 ounces = 1 pint.
8 pints = 1 gallon.

Miscellaneous.

Butt of Sherry	= 108 gals.	Puncheon of Brandy,	110 to 120 gals.
Pipe of Port	= 115 gals.	Puncheon of Rum,	100 to 110 gals.
Butt of Malaga	= 105 gals.	Hogshead of Brandy,	55 to 60 gals.
Puncheon of Scotch Whiskey, 110 to 130 gals.		Hogshead of claret,	46 gals.

Measures of Weight.

The standard avoirdupois pound is the weight of 27.7015 cubic inches of distilled water weighed in air at 39.83°, the barometer at 30 inches.

Avoirdupois, or Ordinary Commercial Weight.

16 drachms	= 1 ounce,	(oz.).
16 ounces	= 1 pound,	(lb.).
100 pounds	= 1 hundred weight	(cwt.).
20 hundred weight	= 1 ton.	

In collecting duties upon foreign goods at the United Sta custom-houses, and also in freighting coal. and selling it by who sale, —

28 pounds	= 1 quarter.
4 quarters, or 112 lbs.	= 1 hundred weight.
20 hundred weight	= 1 long ton = 2240 pounds.
A stone	= 14 pounds.
A quintal	= 100 pounds.

The following measures are sanctioned by custom or law:

32 pounds of oats	= 1 bushel.
45 pounds of Timothy-seed	= 1 bushel.
48 pounds of barley	= 1 bushel.
56 pounds of rye	= 1 bushel.
56 pounds of Indian corn	= 1 bushel.
50 pounds of Indian meal	= 1 bushel.
60 pounds of wheat	= 1 bushel.
60 pounds of clover-seed	= 1 bushel.
60 pounds of potatoes	= 1 bushel.
56 pounds of butter	= 1 firkin.
100 pounds of meal or flour	= 1 sack.
100 pounds of grain or flour	= 1 cental.
100 pounds of dry fish	= 1 quintal.
100 pounds of nails	= 1 cask.
196 pounds of flour	= 1 barrel.
200 pounds of beef or pork	= 1 barrel.

Troy Weight.

USED IN WEIGHING GOLD OR SILVER.

24 grains	= 1 pennyweight	(pwt.).
20 pennyweights	= 1 ounce	(oz.).
12 ounces	= 1 pound	(lb.).

A carat of the jewellers, for precious stones, is, in the Uni States, 3.2 grains: in London, 3.17 grains, in Paris, 3.18 grains divided into 4 jewellers' grains. In troy, apothecaries', and av dupois weights, the grain is the same.

Apothecaries' Weight.

USED IN COMPOUNDING MEDICINES, AND IN PUTTING UP MEDICAL PRESCRIPTIONS.

20 grains (gr.) = 1 scruple (℈).	8 drachms = 1 ounce (oz.).
3 scruples = 1 drachm (ʒ).	12 ounces = 1 pound (lb.).

Measures of Value.

UNITED STATES STANDARD.

10 mills = 1 cent.	10 dimes = 1 dollar.
10 cents = 1 dime.	10 dollars = 1 eagle.

The *standard* of gold and silver is 900 parts of pure metal and 100 of alloy in 1000 parts of coin.

The *fineness* expresses the quantity of pure metal in 1000 parts.

The remedy of the mint is the allowance for deviation from the exact standard fineness and weight of coins.

Weight of Coin.

Double eagle	= 516	troy grains.
Eagle	= 258	troy grains.
Dollar (gold)	= 25.8	troy grains.
Dollar (silver)	= 412.5	troy grains.
Half-dollar	= 192	troy grains.
5-cent piece (nickel)	= 77.16	troy grains.
3-cent piece (nickel)	= 30	troy grains.
Cent (bronze)	= 48	troy grains.

Measure of Time.

60 seconds = 1 minute.	365 days = 1 common year.
60 minutes = 1 hour.	366 days = 1 leap year.
24 hours = 1 day.	

A *solar day* is measured by the rotation of the earth upon its axis with respect to the sun.

In *astronomical computation* and in *nautical time* the day commences at noon, and in the former it is counted throughout the 24 hours.

In *civil computation* the day commences at midnight, and is divided into two portions of 12 hours each.

A solar year is the time in which the earth makes one revolution around the sun; and its average time, called the *mean solar year*, is 365 days, 5 hours, 48 minutes, 49.7 seconds, or nearly 365¼ days.

A mean lunar month, or lunation of the moon, is 29 days, 12 hours, 44 minutes, 2 seconds, and 5.24 thirds.

The Calendar, Old and New Style.

The *Julian* Calendar was established by Julius Cæsar, 44 B.C., and by it one day was inserted in every fourth year. This was the same thing as assuming that the length of the solar year was 365 days, 6 hours, instead of the value given above, thus introducing an accumulative error of 11 minutes, 12 seconds, every year. This calendar was adopted by the church in 325 A.D., at the Council of Nice. In the year 1582 the annual error of 11 minutes, 12 seconds, had amounted to a period of 10 days, which, by order of Pope Gregory XIII., was suppressed in the calendar, and the 5th of October reckoned as the 15th. To prevent the repetition of this error, it was decided to leave out three of the inserted days every 400 years, and to make this omission in the years which are not exactly divisible by 400. Thus, of the years 1700, 1800, 1900, 2000, all of which are leap years according to the Julian Calendar, only the last is a leap year according to the *Reformed* or *Gregorian* Calendar. This Reformed Calendar was not adopted by England until 1752, when 11 days were omitted from the calendar. The two calendars are now often called the *Old Style* and the *New Style*.

The latter style is now adopted in every Christian country except Russia.

Circular and Angular Measures.

USED FOR MEASURING ANGLES AND ARCS, AND FOR DETERMINING LATITUDE AND LONGITUDE.

60 seconds (″)	=	1 minute	(′).
60 minutes	=	1 degree	(°).
360 degrees	=	1 circumference	(C.).

Seconds are usually subdivided into tenths and hundredths.

A *minute* of the circumference of the earth is a geographical mile.

Degrees of the earth's circumference on a meridian average 69.16 common miles.

THE METRIC SYSTEM.

The metric system is a system of weights and measures based upon a unit called a meter.

The *meter* is one ten-millionth part of the distance from the equator to either pole, measured on the earth's surface at the level of the sea.

The *names* of derived metric denominations are formed by prefixing to the name of the primary unit of a measure —

Milli (mill′e), a thousandth,	Hecto (hek′to), one hundred,
Centi (sent′e), a hundredth,	Kilo (kil′o), a thousand,
Deci (des′e), a tenth,	Myria (mir′ea), ten thousand.
Deka (dek′a), ten,	

This system, first adopted by France, has been extensively adopted by other countries, and is much used in the sciences and the arts. It was legalized in 1866 by Congress to be used in the United States, and is already employed by the Coast Survey, and, to some extent, by the Mint and the General Post-Office.

Linear Measures.

The *meter* is the primary unit of lengths.

TABLE.

10 millimeters (mm.)	= 1 centimeter (cm.)	=	0.3937 in.
10 centimeters	= 1 decimeter	=	3.937 in.
10 decimeters	= 1 METER	=	39.37 in.
10 meters	= 1 dekameter	=	393.37 in.
10 dekameters	= 1 hectometer	=	328 ft. 1 in.
10 hectometers	= 1 KILOMETER (km.)	=	0.62137 mi.
10 kilometers	= 1 myriameter	=	6.2137 mi.

The *meter* is used in ordinary measurements; the *centimeter* or *millimeter*, in reckoning very small distances; and the *kilometer*, for roads or great distances.

A *centimeter* is about $\frac{3}{8}$ of an inch; a *meter* is about 3 feet 3 inches and $\frac{3}{8}$; a *kilometer* is about 200 rods, or $\frac{5}{8}$ of a mile.

Surface Measures.

The *square meter* is the primary unit of ordinary surfaces.

The *are* (air), a square, each of whose sides is ten *meters*, is the unit of land measures.

TABLE.

100 square millimeters (sq. mm.)	= 1 square centimeter (sq. cm.)	= 0.155 sq. inch.
100 square centimeters	= 1 square decimeter	= 15.5 sq. inches.
100 square decimeters	= 1 square METER (sq. m.)	= 1550 sq. in., or 1.196 sq. yds.

ALSO

100 centiares, or sq. meters, = 1 ARE (
100 ares = 1 h

A *square meter*, or one *centiaré*, i
square yards, and a *hectare* is about 2½ a

Cubic Measur

The *cubic meter*, or *stere* (stair), is the

TABLE.

1000 cubic millimeters (cu. mm.) = 1 cul

1000 cubic centimeters = 1 cubic decime
1000 cubic decimeters = 1 cubic METER

The *stere* is the name given to the
wood and timber. A tenth of a stere is
are a *dekastere*.

A *cubic meter*, or *stere*, is about 1⅓ cub
feet.

Liquid and Dry M

The *liter* (leeter) is the primary unit
and is a cube, each of whose edges is a t

The *hectoliter* is the unit in measuring
fruits, roots, and liquids.

TABLE.

10 milliliters (ml.)	= 1 centiliter (cl.)
10 centiliters	= 1 deciliter
10 deciliters	= 1 LITER (l.)
10 liters	= 1 dekaliter
10 dekaliters	= 1 HECTOLITER (hl.
10 hectoliters	= 1 kiloliter

A *centiliter* is about ⅓ of a fluid ounce; a *liter* is about $1\frac{1}{16}$ liquid quarts, or $\frac{9}{10}$ of a dry quart; a *hectoliter* is about 2¾ bushels; and a *kiloliter* is one cubic meter, or stere.

Weights.

The *gram* is the primary unit of weights, and is the weight in a vacuum of a cubic centimeter of distilled water at the temperature of 39.2 degrees Fahrenheit.

TABLE.

10 milligrams (mg.)	= 1 centigram	=	0.1543	troy grain.
10 centigrams	= 1 decigram	=	1.543	troy grains.
10 decigrams	= 1 GRAM (g.)	=	15.432	troy grains.
10 grams	= 1 dekagram	=	0.3527	avoir. ounce.
10 dekagrams	= 1 hectogram	=	3.5274	avoir. ounces.
10 hectograms	= 1 KILOGRAM (k.)	=	2.2046	avoir. pounds.
10 kilograms	= 1 myriagram	=	22.046	avoir. pounds.
10 myriagrams	= 1 quintal	=	220.46	avoir. pounds.
10 quintals	= 1 TONNEAU (t.)	=	2204.6	avoir. pounds.

The *gram* is used in weighing gold, jewels, letters, and small quantities of things. The *kilogram*, or, for brevity, *kilo*, is used by grocers; and the *tonneau* (tonno), or *metric ton*, is used in finding the weight of very heavy articles.

A *gram* is about 15½ grains troy; the *kilo* about 2½ pounds avoirdupois; and the *metric ton*, about 2205 pounds.

A *kilo* is the weight of a liter of water at its greatest density; and the *metric ton*, of a cubic meter of water.

Metric numbers are written with the decimal-point (.) at the right of the figures denoting the unit; thus, 15 meters and 3 centimeters are written, 15.03 m.

When metric numbers are expressed by figures, the part of the expression at the left of the decimal-point is read as the number of the unit, and the part at the right, if any, as a number of the lowest denomination indicated, or as a decimal part of the unit; thus, 46.525 m. is read 46 meters and 525 millimeters, or 46 and 525 thousandths meters.

In writing and reading metric numbers, according as the scale is 10, 100, or 1000, each denomination should be allowed one, two, or three orders of figures.

SCRIPTURE AND ANCIENT MEASURES AND WEIGHTS.

Scripture Long Measures.

	Inches.		*Feet.*	*Inches.*
Digit	= 0.912	Cubit	= 1	9.888
Palm	= 3.648	Fathom	= 7	3.552
Span	= 10.944			

Egyptian Long Measures.

Nahud cubit = 1 foot 5.71 inches. Royal cubit = 1 foot 8.66 inches.

Grecian Long Measures.

		Feet.	Inches.			Feet.	Inches.
Digit	=		[illegible]	Stadium	=	[illegible]	4.5
Pous, foot	=	1	[illegible]	Mile	=	[illegible]	
Cubit	=	1	[illegible]				

Jewish Long Measures.

Cubit	= [illegible] ft.	Mile	=	[illegible] feet.
Sabbath-day's journey	= [illegible] ft.	Day's journey	=	[illegible].164 miles.

Roman Long Measures.

		Inches.			Feet.	Inches.
Digit	=	[illegible]	Cubit	=	1	5.406
Uncia, inch	=	[illegible]	Passus	=	4	10.02
Pes, foot	=	[illegible]	Mile, milliarium	=	[illegible]	

Roman Weight.

Ancient libra = [illegible] pound.

Ancient Weights.

		Troy grains.			Troy Grains.
Attic obolus	=	[illegible]	Alexandrian mina	=	9.992
		[illegible]	Denarius Roman	=	51.9
		[illegible]			62.5
Attic drachma	=	[illegible]	Denarius Nero	=	54
		[illegible]	Ounce	=	415.1
Egyptian mina	=	[illegible]			437.2
Ptolemaic mina	=	[illegible]			431.2
Lesser mina	=	[illegible]	Drachm	=	146.5
Greater mina	= [illegible]				
Talent	= [illegible] pounds avoirdupois.				
Pound	= 12 Roman ounces.				

[illegible] for the same weight, [illegible] subject.

Miscellaneous.

		Feet.			Feet.
Arabian foot	=	[illegible]	Hebrew foot	=	1.212
Babylonian foot	=	1.140	Hebrew cubit	=	1.817
Egyptian finger	=	0.06145	Hebrew sacred cubit	=	2.002

MENSURATION.

Definitions.

A *point* is that which has only position.

A *plane* is a surface in which, any two points being taken, the straight line joining them will be wholly in the surface.

Fig. 1
A Curved Line.

A *curved line* is a line of which no portion is straight (Fig. 1).

Parallel lines are such as are wholly in the same plane, and have the same direction (Fig. 2).

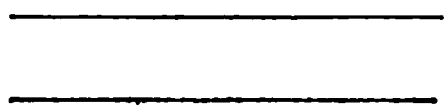


Fig. 2
Parallel Lines.

A *broken line* is a line composed of a series of dashes; thus, —— —— —— ——.

An *angle* is the opening between two lines meeting at a point, and is termed a *right angle* when the two lines are perpendicular to each other, an *acute angle* when it is less or sharper than a right angle, and *obtuse* when it is greater than a right angle. Thus, in Fig. 3,

A A A A are *acute angles*,
O O O O are *obtuse angles*,
R R R R are *right angles*.

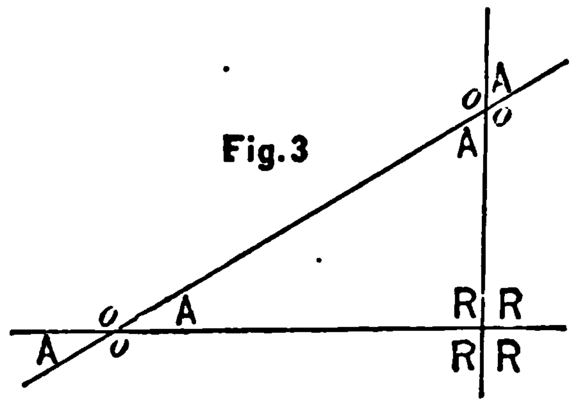


Fig. 3

Polygons.

A *polygon* is a portion of a plane bounded by straight lines.

A *triangle* is a polygon of three sides.

A *scalene triangle* has none of its sides equal; an *isosceles triangle* has two of its sides equal; an *equilateral triangle* has all three of its sides equal.

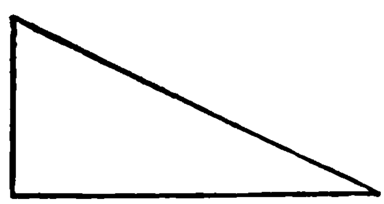


Fig. 4.
Right-angle Triangle.

A *right-angle triangle* is one which has a right angle. The side opposite the right angle is called the *hypothenuse;* the side on which the triangle is supposed to stand is called its *base*, and the other side, its *altitude*.

Fig. 5.
Scalene Triangle.

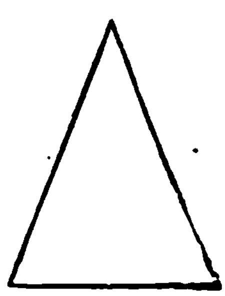


Fig. 6.
Isosceles Triangle.

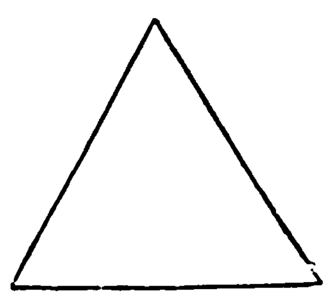


Fig. 7.
Equilateral Triangle.

A *quadrilateral* is a polygon of four sides.

Quadrilaterals are divided into classes, as follows,—the *trapezium* (Fig. 8), which has no two of its sides parallel; the *trapezoid* (Fig. 9), which has two of its sides parallel; and the *parallelogram* (Fig. 10), which is bounded by two pairs of parallel sides.

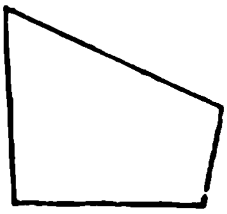


Fig. 8.

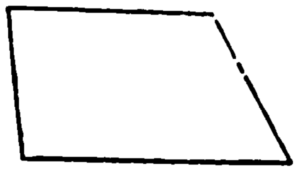


Fig. 9.

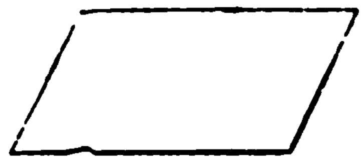


Fig. 10.

A parallelogram whose sides are not equal, and its angles not right angles, is called a *rhomboid* (Fig. 11); when the sides are all equal, but the angles are not right angles, it is called a *rhombus* (Fig. 12); and, when the angles are right angles, it is called a *rectangle* (Fig. 13). A rectangle whose sides are all equal is called a *square* (Fig. 14). Polygons whose sides are all equal are called *regular*.

Fig. 11.

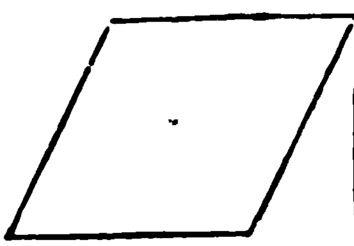


Fig. 12.

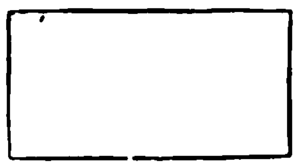


Fig. 13.

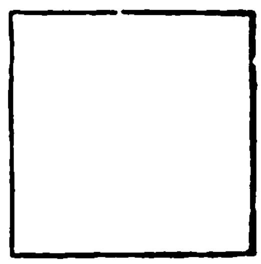


Fig. 14.

Besides the square and equilateral triangles, there are

The *pentagon* (Fig. 15), which has five sides;
The *hexagon* (Fig. 16), which has six sides;
The *heptagon* (Fig. 17), which has seven sides;
The *octagon* (Fig. 18), which has eight sides.

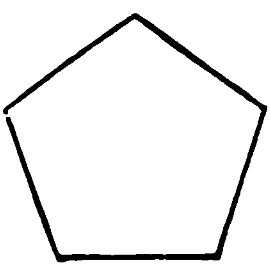


Fig. 15.

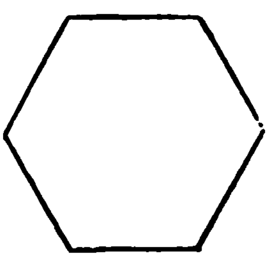


Fig. 16.

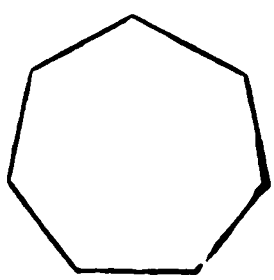


Fig. 17.

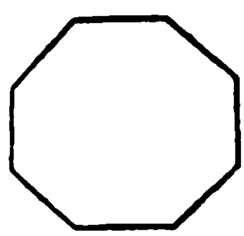


Fig. 18.

The *enneagon* has nine sides.
The *decagon* has ten sides.
The *dodecagon* has twelve sides.

For all polygons, the side upon which it is supposed to stand is called its *base*; the perpendicular distance from the highest side or

angle to the base (prolonged, if necessary) is called the *altitude*; and a line joining any two angles not adjacent is called a *diagonal.*

A *perimeter* is the boundary line of a plane figure.

A *circle* is a portion of a plane bounded by a curve, all the points of which are equally distant from a point within called the centre (Fig. 19).

The *circumference* is the curve which bounds the circle.

A *radius* is any straight line drawn from the centre to the circumference.

Any straight line drawn through the centre to the circumference on each side is called a *diameter.*

An *arc* of a circle is any part of its circumference.

A *chord* is any straight line joining two points of the circumference, as *bd.*

A *segment* is a portion of the circle included between the arc and its chord, as A in Fig. 19.

A *sector* is the space included between an arc and two radii drawn to its extremities, as B, Fig. 19. In the figure, *ab* is a radius, *cd* a diameter, and *db* is a chord *subtending* the arc *bcd.* A *tangent* is a right line which in passing a curve touches without cutting it, as *fg*, Fig. 19.

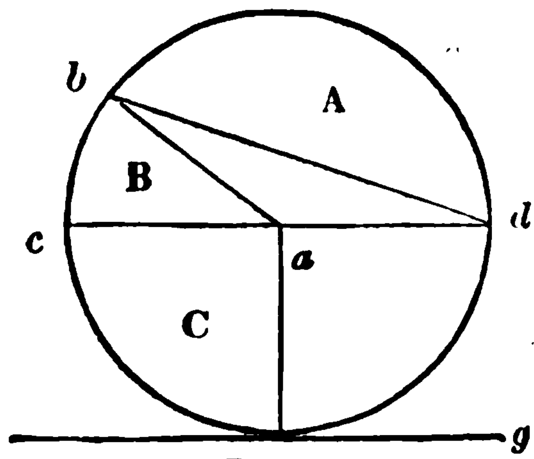


Fig. 19.

Volumes.

A *prism* is a volume whose ends are equal and parallel polygons, and whose sides are parallelograms.

A prism is *triangular*, *rectangular*, etc., according as its ends are *triangles*, *rectangles*, etc.

A *cube* is a rectangular prism all of whose sides are squares.

A *cylinder* is a volume of uniform diameter, bounded by a curved surface and two equal and parallel circles.

A *pyramid* is a volume whose base is a polygon, and whose sides are triangles meeting in a point called the *vertex.*

A pyramid is triangular, quadrangular, etc., according as its base is a triangle, quadrilateral, etc.

A *cone* is a volume whose base is a circle, from which the remaining surface tapers uniformly to a point or vertex (Fig. 20).

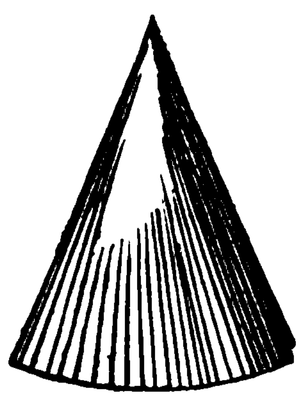


Fig. 20.

Conic sections are the figures made by a plane cutting a cone.

An *ellipse* is the section of a cone when cut by a plane passing obliquely through both sides, as at *ab*, Fig. 21.

A *parabola* is a section of a cone cut by a plane parallel to its side, as at *cd*.

A *hyperbola* is a section of a cone cut by a plane at a greater angle through the base than is made by the side of the cone, as at *eh*.

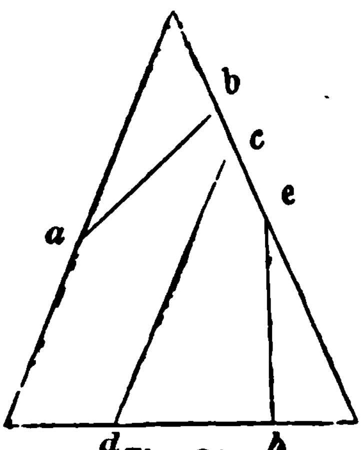


Fig. 21.

In the ellipse, the *transverse axis*, or *long diameter*, is the longest line that can be drawn through it. The *conjugate axis*, or *short diameter*, is a line drawn through the centre, at right angles to the long diameter.

A *frustum of a pyramid* or *cone* is that which remains after cutting off the upper part of it by a plane parallel to the base.

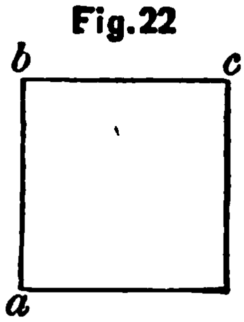
A *sphere* is a volume bounded by a curved surface, all points of which are equally distant from a point within, called the centre.

Mensuration *treats of the measurement of lines, surfaces, and volumes.*

RULES.

To compute the area of a square, a rectangle, a rhombus, or a rhomboid.

RULE. — Multiply the length by the breadth or height; thus, in either of Figs. 22, 23, 24, the area = $ab \times bc$.

Fig.22


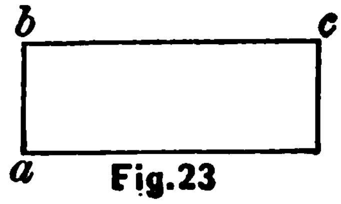


Fig.23

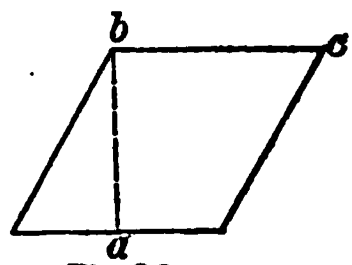


Fig.24

To compute the area of a triangle.

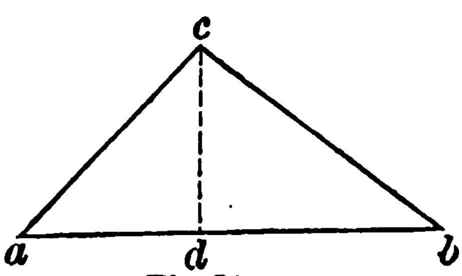


Fig.25

RULE. — Multiply the base by the altitude, and divide by 2; thus, in Fig. 25,

area of $abc = \dfrac{ab \times cd}{2}$.

To find the length of the hypothenuse of a right-angle triangle when both sides are known.

RULE. — Square the length of each of the sides making the right angle, add their squares together, and take the square root of their sum. Thus (Fig. 26), the length of $ac = 3$, and of $bc = 4$; then

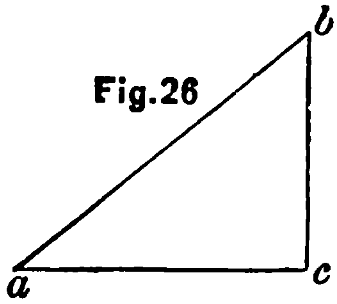


Fig.26

$$ab = 3 \times 3 = 9 + (4 \times 4) = 9 + 16 = 25.$$

$$\sqrt{25} = 5, \quad \text{or} \quad ab = 5.$$

To find the length of the base or altitude of a right-angle triangle, when the length of the hypothenuse and one side is known.

RULE. — From the square of the length of the hypothenuse subtract the square of the length of the other side, and take the square root of the remainder.

To find the area of a trapezium.

RULE. — Multiply the diagonal by the sum of the two perpendiculars falling upon it from the opposite angles, and divide the product by 2. Or,

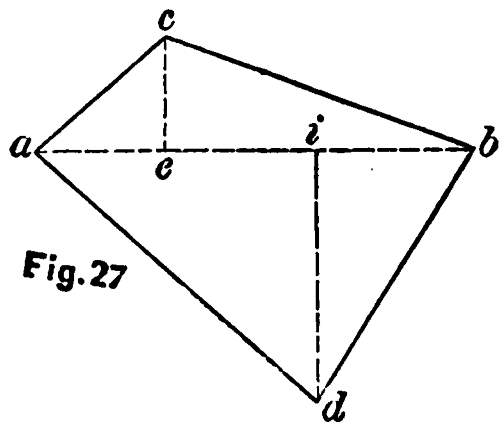


Fig. 27

$$\frac{ab \times (ce + di)}{2} = \text{area (Fig. 27).}$$

To find the area of a trapezoid (Fig. 28).

RULE. — Multiply the sum of the two parallel sides by the perpendicular distance between them, and divide the product by 2.

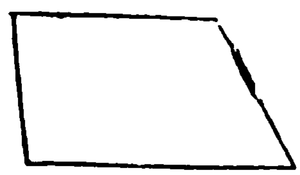
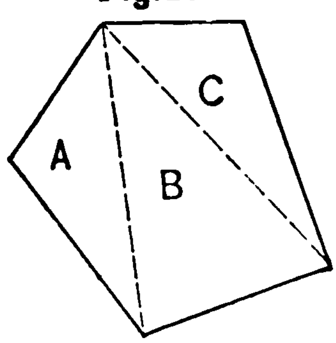


Fig. 28.

To compute the area of an irregular polygon.

Fig. 29



RULE. — Divide the polygon into triangles by means of diagonal lines, and then add together the areas of all the triangles, as A, B, and C (Fig. 29).

To find the area of a regular polygon.

RULE. — Multiply the length of a side by the perpendicular distance to the centre (as ao, Fig. 30), and that product by the number of sides, and divide the result by 2.

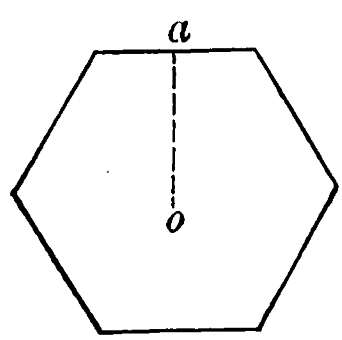


Fig. 30

To compute the area of a regular polygon when the length of a side only is given.

RULE. — Multiply the square of the side by the multiplier opposite to the name of the polygon in column A of the following table: —

Name of Polygon.	No. of sides.	A. Area.	B. Radius of circumscribing circle.	C. Length of the side.	D. Radius of inscribed circle.
Triangle . . .	3	0.433013	0.5773	1.732	0.2887
Tetragon . .	4	1	0.7071	1.4142	0.5
Pentagon . . .	5	1.720477	0.8506	1.1756	0.6882
Hexagon . . .	6	2.598076	1	1	0.866
Heptagon . . .	7	3.633912	1.1524	0.8677	1.0383
Octagon . . .	8	4.828427	1.3066	0.7653	1.2071
Nonagon . . .	9	6.181824	1.4619	0.684	1.3737
Decagon . . .	10	7.694209	1.618	0.618	1.5383
Undecagon . .	11	9.36564	1.7747	0.5634	1.7028
Dodecagon . .	12	11.196152	1.9319	0.5176	1.866

To compute the radius of a circumscribing circle when the length of a side only is given.

RULE. — Multiply the length of a side of the polygon by the number in column *B*.

EXAMPLE. — What is the radius of a circle that will contain a hexagon, the length of one side being 5 inches?

Ans. $5 \times 1 = 5$ inches.

To compute the length of a side of a polygon that is contained in a given circle, when the radius of the circle is given.

RULE. — Multiply the radius of the circle by the number opposite the name of the polygon in column *C*.

EXAMPLE. — What is the length of the side of a pentagon contained in a circle 8 feet in diameter?

Ans. 8 ft. diameter $\div 2 = 4$ ft. radius, $4 \times 1.1756 = 4.7024$ ft.

To compute the radius of a circle that can be inscribed in a given polygon, when the length of a side is given.

RULE. — Multiply the length of a side of the polygon by the number opposite the name of the polygon in column *D*.

EXAMPLE. — What is the radius of the circle that can be inscribed in an octagon, the length of one side being 6 inches.

Ans. $6 \times 1.2071 = 7.2426$ inches.

Circles.

To compute the circumference of a circle.

RULE. — Multiply the diameter by 3.1416; or, for most purposes, by $3\frac{1}{7}$ is sufficiently accurate.

EXAMPLE. — What is the circumference of a circle 7 inches in diameter?

Ans. $7 \times 3.1416 = 21.9912$ inches, or $7 \times 3\frac{1}{7} = 22$ inches, the error in this last being 0.0088 of an inch.

To find the diameter of a circle when the circumference is given.

RULE.—Divide the circumference by 3.1416, or for a very near approximate result multiply by 7 and divide by 22.

To find the radius of an arc, when the chord and rise or versed sine are given.

RULE.—Square *one-half* the *chord*, also square the *rise;* divide their sum by twice the rise; the result will be the radius.

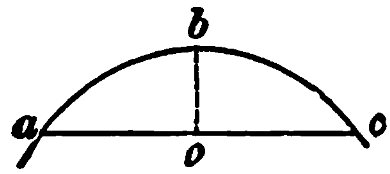


Fig. 30½.

EXAMPLE.—The length of the chord *ac*, Fig. 30½, is 48 inches, and the rise, *bo*, is 6 inches. What is the radius of the arc?

Ans. Rad $= \frac{oc^2 + bo^2}{2bo} = \frac{24^2 + 6^2}{12} = 51$ ins.

To find the rise or versed sine of a circular arc, when the chord and radius are given.

RULE.—Square the radius; also square one-half the chord; subtract the latter from the former, and take the square root of the remainder. Subtract the result from the radius, and the remainder will be the rise.

EXAMPLE.—A given chord has a radius of 51 inches, and a chord of 48 inches. What is the rise?

Ans. Rise $= \text{rad} - \sqrt{\text{rad}^2 - \tfrac{1}{2}\text{chord}^2} = 51 - \sqrt{2601 - 576}$
$= 51 - 45 = 6$ inches = rise.

To compute the area of a circle.

RULE.—Multiply the square of the diameter by 0.7854, or multiply the square of the radius by 3.1416.

EXAMPLE.—What is the area of a circle 10 inches in diameter?

Ans. $10 \times 10 \times 0.7854 = 78.54$ square inches, or $5 \times 5 \times 3.1416$ $= 78.54$ square inches.

The following tables will be found very convenient for finding the circumference and area of circles.

AREAS AND CIRCUMFERENCES OF CIRCLES

(Advancing by Tenths.)

Diam.	Area.	Circum.	Diam.	Area.	Circum.	Diam.	Area.	Circum.
30.0	706.8583	94.2478	35.0	962.1128	109.9557	40.0	1256.6371	125.6637
.1	711.5786	94.5619	.1	967.6184	110.2699	.1	1262.9281	125.9779
.2	716.3145	94.8761	.2	973.1397	110.5841	.2	1269.2348	126.2920
.3	721.0662	95.1903	.3	978.6768	110.8982	.3	1275.5573	126.6062
.4	725.8336	95.5044	.4	984.2296	111.2124	.4	1281.8955	126.9203
.5	730.6167	95.8186	.5	989.7980	111.5265	.5	1288.2493	127.2345
.6	735.4154	96.1327	.6	995.3822	111.8407	.6	1294.6189	127.5487
.7	740.2299	96.4469	.7	1000.9821	112.1549	.7	1301.0042	127.8628
.8	745.0601	96.7611	.8	1006.5977	112.4690	.8	1307.4052	128.1770
.9	749.9060	97.0752	.9	1012.2290	112.7832	.9	1313.8219	128.4911
31.0	754.7676	97.3894	36.0	1017.8760	113.0973	41.0	1320.2543	128.8053
.1	759.6450	97.7035	.1	1023.5387	113.4115	.1	1326.7024	129.1195
.2	764.5380	98.0177	.2	1029.2172	113.7257	.2	1333.1663	129.4336
.3	769.4467	98.3319	.3	1034.9113	114.0398	.3	1339.6458	129.7478
.4	774.3712	98.6460	.4	1040.6212	114.3540	.4	1346.1410	130.0619
.5	779.3113	98.9602	.5	1046.3467	114.6681	.5	1352.6520	130.3761
.6	784.2672	99.2743	.6	1052.0880	114.9823	.6	1359.1786	130.6903
.7	789.2388	99.5885	.7	1057.8449	115.2965	.7	1365.7210	131.0044
.8	794.2260	99.9026	.8	1063.6176	115.6106	.8	1372.2791	131.3186
.9	799.2290	100.2168	.9	1069.4060	115.9248	.9	1378.8529	131.6327
32.0	804.2477	100.5310	37.0	1075.2101	116.2389	42.0	1385.4424	131.9469
.1	809.2821	100.8451	.1	1081.0299	116.5531	.1	1392.0476	132.2611
.2	814.3322	101.1593	.2	1086.8654	116.8672	.2	1398.6685	132.5752
.3	819.3980	101.4734	.3	1092.7166	117.1814	.3	1405.3051	132.8894
.4	824.4796	101.7876	.4	1098.5835	117.4956	.4	1411.9574	133.2035
.5	829.5768	102.1018	.5	1104.4662	117.8097	.5	1418.6254	133.5177
.6	834.6898	102.4159	.6	1110.3645	118.1239	.6	1425.3092	133.8318
.7	839.8185	102.7301	.7	1116 2786	118.4380	.7	1432.0086	134.1460
.8	844.9628	103.0442	.8	1122.2083	118.7522	.8	1438.7238	134.4602
.9	850.1229	103.3584	.9	1128.1538	119.0664	.9	1445.4546	134.7743
33.0	855.2986	103.6726	38.0	1134.1149	119.3805	43.0	1452.2012	135.0885
.1	860.4902	103.9867	.1	1140.0918	119.6947	.1	1458.9635	135.4026
.2	865.6973	104.3009	.2	1146.0844	120.0088	.2	1465.7415	135.7168
.3	870.9202	104.6150	.3	1152.0927	120.3230	.3	1472.5352	136.0310
.4	876.1588	104.9292	.4	1158.1167	120.6372	.4	1479.3446	136.3451
.5	881.4131	105.2434	.5	1164.1564	120.9513	.5	1486.1697	136.6593
.6	886.6831	105.5575	.6	1170.2118	121.2655	.6	1493.0105	136.9734
.7	891.9688	105.8717	.7	1176.2830	121.5796	.7	1499.8670	137.2876
.8	897.2703	106.1858	.8	1182.3698	121.8938	.8	1506.7393	137.6018
.9	902.5874	106.5000	.9	1188.4724	122.2080	.9	1513.6272	137.9159
34.0	907.9203	106.8142	39.0	1194.5906	122.5221	44.0	1520.5308	138.2301
.1	913.2688	107.1283	.1	1200.7246	122.8363	.1	1527.4502	138.5442
.2	918.6331	107.4425	.2	1206.8742	123.1504	.2	1534.3853	138.8584
.3	924.0131	107.7566	.3	1213.0396	123.4646	.3	1541.3360	139.1726
.4	929.4088	108.0708	.4	1219.2207	123.7788	.4	1548.3025	139.4867
.5	934.8202	108.3849	.5	1225.4175	124.0929	.5	1555.2847	139.8009
.6	940.2473	108.6991	.6	1231.6300	124.4071	.6	1562.2826	140.1153
.7	945.6901	109.0133	.7	1237.8582	124.7212	.7	1569.2962	140.4292
.8	951.1486	109.3274	.8	1244.1021	125.0354	.8	1576.3255	140.7434
.9	956.6228	109.6416	.9	1250.3617	125.3495	.9	1583.3706	141.0575

AREAS AND CIRCUMFERENCES OF CIRCLES.

(Advancing by Tenths.)

AREAS AND CIRCUMFERENCES OF CIRCLES.

(Advancing by Tenths.)

.3	3247.2222	202.0044	.3	3771.8668	217.7124	.3	4335.7827	233.4203
.4	3257.3289	202.3186	.4	3782.7603	218.0265	.4	4347.4616	233.7345
.5	3267.4527	202.6327	.5	3793.6695	218.3407	.5	4359.1663	234.0487
.6	3277.5922	202.9469	.6	3804.5944	218.6548	.6	4370.8564	234.3628
.7	3287.7474	203.2610	.7	3815.5350	218.9690	.7	4382.5924	234.6770
.8	3297.9183	203.5752	.8	3826.4913	219.2832	.8	4394.3841	234.9911
.9	3308.1049	203.8894	.9	3837.4633	219.5973	.9	4406.0916	235.3053

AREAS AND CIRCUMFERENCES OF CIRCLES.

(*Advancing by Tenths.*)

AREAS AND CIRCUMFERENCES OF CIRCLES.

(Advancing by Tenths.)

Diam	Area.	Circum.	Diam.	Area.	Circum.	Diam.	Area.	Circum.
90.0	6361.7251	282.7433	93.5	6866.1471	293.7389	97.0	7389.8113	304.7345
.1	6375.8701	283.0575	.6	6880.8419	294.0531	.1	7405.0559	305.0486
.2	6390.0309	283.3717	.7	6895.5524	294.3672	.2	7420.3162	305.3628
.3	6404.2073	283.6858	.8	6910.2786	294.6814	.3	7435.5922	305.6770
.4	6418.3995	284.0000	.9	6925.0205	294.9956	.4	7450.8839	305.9911
.5	6432.6073	284.3141	94.0	6939.7782	295.3097	.5	7466.1913	306.3053
.6	6446.8309	284.6283	.1	6954.5515	295.6239	.6	7481.5144	306.6194
.7	6461.0701	284.9425	.2	6969.3106	295.9380	.7	7496.8532	306.9336
.8	6475.3251	285.2566	.3	6984.1453	296.2522	.8	7512.2078	307.2478
.9	6489.5958	285.5708	.4	6998.9658	296.5663	.9	7527.5780	307.5619
91.0	6503.8822	285.8849	.5	7013.8019	296.8805	98.0	7542.9640	307.8761
.1	6518.1843	286.1991	.6	7028.6538	297.1947	.1	7558.3656	308.1902
.2	6532.5021	286.5133	.7	7043.5214	297.5088	.2	7573.7830	308.5044
.3	6546.8356	286.8274	.8	7058.4047	297.8230	.3	7589.2161	308.8186
.4	6561.1848	287.1416	.9	7073.3033	298.1371	.4	7604.6648	309.1327
.5	6575.5498	287.4557	95.0	7088.2184	298.4513	.5	7620.1293	309.4469
.6	6589.9304	287.7699	.1	7103.1488	298.7655	.6	7635.6095	309.7610
.7	6604.3268	288.0840	.2	7118.1950	299.0796	.7	7651.1054	310.0752
.8	6618.7388	288.3982	.3	7133.0568	299.3938	.8	7666.6170	310.3894
.9	6633.1666	288.7124	.4	7148.0343	299.7079	.9	7682.1444	310.7035
92.0	6647.6101	289.0265	.5	7163.0276	300.0221	99.0	7697.6893	311.0177
.1	6662.0692	289.3407	.6	7178.0366	300.3363	.1	7713.2461	311.3318
.2	6676.5441	289.6548	.7	7193.0612	300.6504	.2	7728.8206	311.6460
.3	6691.0347	289.9690	.8	7208.1016	300.9646	.3	7744.4107	311.9602
.4	6705.5410	290.2832	.9	7223.1577	301.2787	.4	7760.0166	312.2743
.5	6720.0630	290.5973	96.0	7238.2295	301.5929	.5	7775.6382	312.5885
.6	6734.6008	290.9115	.1	7253.3170	301.9071	.6	7791.2754	312.9026
.7	6749.1542	291.2256	.2	7268.4202	302.2212	.7	7806.9284	313.2168
.8	6763.7233	291.5398	.3	7283.5391	302.5354	.8	7822.5971	313.5309
.9	6778.3082	291.8540	.4	7298.6737	302.8405	.9	7838.2815	313.8451
93.0	6792.9087	292.1681	.5	7313.8240	303.1637	100.0	7853.9816	314.1593
.1	6807.5250	292.4823	.6	7328.9901	303.4779			
.2	6822.1569	292.7964	.7	7344.1718	303.7920			
.3	6836.8046	293.1106	.8	7359.3693	304.1062			
.4	6851.4680	293.4248	.9	7374.5824	304.4203			

AREAS OF CIRCLES.

(ADVANCING BY EIGHTHS.)

AREAS.

Diam.	0.0	$0.\frac{1}{8}$	$0.\frac{1}{4}$	$0.\frac{3}{8}$	$0.\frac{1}{2}$	$0.\frac{5}{8}$	$0.\frac{3}{4}$	$0.\frac{7}{8}$
0	0.0	0.0122	0.0490	0.1104	0.1963	0.3068	0.4417	0.6013
1	0.7854	0.9940	1.227	1.484	1.767	2.073	2.405	2.761
2	3.1416	3.546	3.976	4.430	4.908	5.411	5.939	6.491
3	7.068	7.669	8.295	8.946	9.621	10.32	11.04	11.79
4	12.56	13.36	14.18	15.03	15.90	16.80	17.72	18.66
5	19.63	20.62	21.64	22.69	23.75	24.85	25.96	27.10
6	28.27	29.46	30.67	31.91	33.18	34.47	35.78	37.12
7	38.48	39.87	41.28	42.71	44.17	45.66	47.17	48.70
8	50.26	51.84	53.45	55.08	56.74	58.42	60.13	61.86
9	63.61	65.39	67.20	69.02	70.88	72.75	74.66	76.58
10	78.54	80.51	82.51	84.54	86.59	88.66	90.76	92.88
11	95.03	97.20	99.40	101.6	103.8	106.1	108.4	110.7
12	113.0	115.4	117.8	120.2	122.7	125.1	127.6	130.1
13	132.7	135.2	137.8	140.5	143.1	145.8	148.4	151.2
14	153.9	156.6	159.4	162.2	165.1	167.9	170.8	173.7
15	176.7	179.6	182.6	185.6	188.6	191.7	194.8	197.9
16	201.0	204.2	207.3	210.5	213.8	217.0	220.3	223.6
17	226.9	230.3	233.7	237.1	240.5	243.9	247.4	250.9
18	254.4	258.0	261.5	265.1	268.8	272.4	276.1	279.8
19	283.5	287.2	291.0	294.8	298.6	302.4	306.3	310.2
20	314.1	318.1	322.0	326.0	330.0	334.1	338.1	342.2
21	346.3	350.4	354.6	358.8	363.0	367.2	371.5	375.8
22	380.1	384.4	388.8	393.2	397.6	402.0	406.4	410.9
23	415.4	420.0	424.5	429.1	433.7	438.3	443.0	447.6
24	452.3	457.1	461.8	466.6	471.4	476.2	481.1	485.9
25	490.8	495.7	500.7	505.7	510.7	515.7	520.7	525.8
26	530.9	536.0	541.1	546.3	551.5	556.7	562.0	567.2
27	572.5	577.8	583.2	588.5	593.9	599.3	604.8	610.2
28	615.7	621.2	626.7	632.3	637.9	643.5	649.1	654.8
29	660.5	666.2	671.9	677.7	683.4	689.2	695.1	700.9
30	706.8	712.7	718.6	724.6	730.6	736.6	742.6	748.6
31	754.8	760.9	767.0	773.1	779.3	785.5	791.7	798.0
32	804.3	810.6	816.9	823.2	829.6	836.0	842.4	848.8
33	855.3	861.8	868.3	874.9	881.4	888.0	894.6	901.3
34	907.9	914.7	921.3	928.1	934.8	941.6	948.4	955.3
35	962.1	969.0	975.9	982.8	989.8	996.8	1003.8	1010.8
36	1017.9	1025.0	1032.1	1039.2	1046.3	1053.5	1060.7	1068.0
37	1075.2	1082.5	1089.8	1097.1	1104.5	1111.8	1119.2	1126.7
38	1134.1	1141.6	1149.1	1156.6	1164.2	1171.7	1179.3	1186.9
39	1194.6	1202.3	1210.0	1217.7	1225.4	1233.2	1241.0	1248.8
40	1256.6	1264.5	1272.4	1280.3	1288.2	1296.2	1304.2	1312.2
41	1320.3	1328.3	1336.4	1344.5	1352.7	1360.8	1369.0	1377.2
42	1385.4	1393.7	1402.0	1410.3	1418.6	1427.0	1435.4	1443.8
43	1452.2	1460.7	1469.1	1477.6	1486.2	1494.7	1503.3	1511.9
44	1520.5	1529.2	1537.9	1546.6	1555.3	1564.0	1572.8	1581.6
45	1590.4	1599.3	1608.2	1617.0	1626.0	1634.9	1643.9	1652.9

CIRCUMFERENCES OF CIRCLES.

(ADVANCING BY EIGHTHS.)

CIRCUMFERENCES.

Diam.	0.0	$0.\frac{1}{8}$	$0.\frac{1}{4}$	$0.\frac{3}{8}$	$0.\frac{1}{2}$	$0.\frac{5}{8}$	$0.\frac{3}{4}$	$0.\frac{7}{8}$
0	0.0	0.3927	0.7854	1.178	1.570	1.963	2.356	2.748
1	3.141	3.534	3.927	4.319	4.712	5.105	5.497	5.890
2	6.283	6.675	7.068	7.461	7.854	8.246	8.639	9.032
3	9.424	9.817	10.21	10.60	10.99	11.38	11.78	12.17
4	12.56	12.95	13.35	13.74	14.13	14.52	14.92	15.31
5	15.70	16.10	16.49	16.88	17.27	17.67	18.06	18.45
6	18.84	19.24	19.63	20.02	20.42	20.81	21.20	21.59
7	21.99	22.38	22.77	23.16	23.56	23.95	24.34	24.74
8	25.13	25.52	25.91	26.31	26.70	27.09	27.48	27.88
9	28.27	28.66	29.05	29.45	29.84	30.23	30.63	31.02
10	31.41	31.80	32.20	32.59	32.98	33.37	33.77	34.16
11	34.55	34.95	35.34	35.73	36.12	36.52	36.91	37.30
12	37.69	38.09	38.48	38.87	39.27	39.66	40.05	40.44
13	40.84	41.23	41.62	42.01	42.41	42.80	43.19	43.58
14	43.98	44.37	44.76	45.16	45.55	45.94	46.33	46.73
15	47.12	47.51	47.90	48.30	48.69	49.08	49.48	49.87
16	50.26	50.65	51.05	51.44	51.83	52.22	52.62	53.01
17	53.40	53.79	54.19	54.58	54.97	55.37	55.76	56.15
18	56.54	56.94	57.33	57.72	58.11	58.51	58.90	59.29
19	59.69	60.08	60.47	60.86	61.26	61.65	62.04	62.43
20	62.83	63.22	63.61	64.01	64.40	64.79	65.18	65.58
21	65.97	66.36	66.75	67.15	67.54	67.93	68.32	68.72
22	69.11	69.50	69.90	70.29	70.68	71.07	71.47	71.86
23	72.25	72.64	73.04	73.43	73.82	74.22	74.61	75.00
24	75.39	75.79	76.18	76.57	76.96	77.36	77.75	78.14
25	78.54	78.93	79.32	79.71	80.10	80.50	80.89	81.28
26	81.68	82.07	82.46	82.85	83.25	83.64	84.03	84.43
27	84.82	85.21	85.60	86.00	86.39	86.78	87.17	87.57
28	87.96	88.35	88.75	89.14	89.53	89.92	90.32	90.71
29	91.10	91.49	91.89	92.28	92.67	93.06	93.46	93.85
30	94.24	94.64	95.03	95.42	95.81	96.21	96.60	96.99
31	97.39	97.78	98.17	98.57	98.96	99.35	99.75	100.14
32	100.53	100.92	101.32	101.71	102.10	102.49	102.89	103.29
33	103.67	104.07	104.46	104.85	105.24	105.64	106.03	106.42
34	106.81	107.21	107.60	107.99	108.39	108.78	109.17	109.56
35	109.96	110.35	110.74	111.13	111.53	111.92	112.31	112.71
36	113.10	113.49	113.88	114.28	114.67	115.06	115.45	115.85
37	116.24	116.63	117.02	117.42	117.81	118.20	118.60	118.99
38	119.38	119.77	120.17	120.56	120.95	121.34	121.74	122.13
39	122.52	122.92	123.31	123.70	124.09	124.49	124.88	125.27
40	125.66	126.06	126.45	126.84	127.24	127.63	128.02	128.41
41	128.81	129.20	127.59	129.98	130.38	130.77	131.16	131.55
42	131.95	132.34	132.73	133.13	133.52	133.91	134.30	134.70
43	135.09	135.48	135.87	136.27	136.66	137.05	137.45	137.84
44	138.23	138.62	139.02	139.41	139.80	140.19	140.59	140.98
45	141.37	141.76	142.16	142.55	142.94	143.34	143.73	144.12

AREAS AND CIRCUMFERENCES OF CIRCLES.

From 1 to 50 Feet.

(ADVANCING BY ONE INCH.)

Diam.	Area.	Circum.	Diam.	Area.	Circum.	Diam.	Area.	Circum.
Ft.	*Feet.*	*Ft. In.*	*Ft.*	*Feet.*	*Ft. In.*	*Ft.*	*Feet.*	*Ft. In.*
1 0	0.7854	3 1⅝	5 0	19.635	15 8½	9 0	63.6174	28 3¼
1	0.9217	3 4⅞	1	20.2947	15 11⅝	1	64.8006	28 6⅜
2	1.069	3 8	2	20.9656	16 2¾	2	65.9951	28 9½
3	1.2271	3 11	3	21.6475	16 5¾	3	67.2007	29 ⅝
4	1.3962	4 2¼	4	22.34	16 9	4	68.4166	29 3¾
5	1.5761	4 5⅜	5	23.0437	17 ⅛	5	69.644	29 7
6	1.7671	4 8½	6	23.7583	17 3⅜	6	70.8823	29 10⅛
7	1.9689	4 11⅝	7	24.4835	17 6½	7	72.1309	30 1¼
8	2.1816	5 2⅞	8	25.2199	17 9⅝	8	73.391	30 4⅜
9	2.4052	5 5⅞	9	25.9672	18 ¾	9	74.662	30 7½
10	2.6398	5 9	10	26.7251	18 3⅞	10	75.9433	30 11⅝
11	2.8852	6 2¼	11	27.4943	18 7⅛	11	77.2362	31 1¾
2 0	3.1416	6 3⅜	6 0	28.2744	18 10⅛	10 0	78.54	31 5
1	3.4087	6 6½	1	29.0649	19 1⅜	1	79.854	31 8⅛
2	3.6869	6 9⅝	2	29.8668	19 4½	2	81.1795	31 11¼
3	3.976	7 ¾	3	30.6796	19 7½	3	82.516	32 2⅜
4	4.276	7 3⅞	4	31.5029	19 10¾	4	83.8627	32 5½
5	4.5869	7 7	5	32.3376	20 1⅞	5	85.2211	32 8⅝
6	4.9087	7 10¼	6	33.1831	20 4⅞	6	86.5903	32 11¾
7	5.2413	8 1⅜	7	34.0391	20 8⅛	7	87.9697	33 2⅞
8	5.585	8 4½	8	34.9065	20 11½	8	89.3608	33 6⅛
9	5.9395	8 7⅝	9	35.7847	21 2½	9	90.7627	33 9¼
10	6.3049	8 10¾	10	36.6735	21 5½	10	92.1749	34 ⅜
11	6.6813	9 1⅞	11	37.5736	21 8¾	11	93.5986	34 3½
3 0	7.0686	9 5	7 0	38.4846	21 11⅞	11 0	95.0334	34 6⅝
1	7.4666	9 8¼	1	39.406	22 3	1	96.4783	34 9¾
2	7.8757	9 11⅜	2	40.3388	22 6⅛	2	97.9347	35 ⅞
3	8.2957	10 2½	3	41.2825	22 9¼	3	99.4021	35 4⅛
4	8.7265	10 5⅝	4	42.2367	23 ⅜	4	100.8797	35 7¼
5	9.1683	10 8¾	5	43.2022	23 2⅛	5	102.3689	35 10⅜
6	9.6211	10 11⅞	6	44.1787	23 6¾	6	103.8691	36 1½
7	10.0846	11 3	7	45.1656	23 9⅞	7	105.3794	36 4½
8	10.5591	11 6¼	8	46.1638	24 1⅛	8	106.9013	36 7¾
9	11.0146	11 9⅜	9	47.173	24 4⅛	9	108.4342	36 10⅞
10	11.5409	12 ½	10	48.1962	24 7¼	10	109.9772	37 2¾
11	12.0481	12 3⅝	11	49.2236	24 10⅜	11	111.5319	37 5¼
4 0	12.5664	12 6¾	8 0	50.2656	25 1½	12 0	113.0976	37 8⅜
1	13.0352	12 9⅞	1	51.3178	25 4⅝	1	114.6732	37 11½
2	13.6353	13 1	2	52.3816	25 7⅞	2	116.2607	38 2⅝
3	14.1862	13 4¼	3	53.4562	25 11	3	117.859	38 5¾
4	14.7479	13 7⅜	4	54.5412	26 2⅛	4	119.4674	38 8⅞
5	15.3206	13 10½	5	55.6377	26 5¼	5	121.0876	39 0
6	15.9043	14 1⅝	6	56.7451	26 8⅜	6	122.7187	39 3¼
7	16.4986	14 4¾	7	57.8628	26 11½	7	124.3598	39 6⅜
8	17.1041	14 7⅞	8	58.992	27 2¾	8	126.0127	39 9½
9	17.7205	14 11	9	60.1321	27 5¾	9	127.6765	40 ⅝
10	18.3476	15 2¼	10	61.2826	27 9	10	129.3504	40 3¾
11	18.9858	15 5⅜	11	62.4445	28 ⅛	11	131.036	40 6⅞

Areas and Circumferences of Circles (Feet and Inches).

Diam.	Area.	Circum.	Diam.	Area.	Circum.	Diam.	Area.	Circum.
Ft.	*Feet.*	*Ft. In.*	*Ft.*	*Feet.*	*Ft. In.*	*Ft.*	*Feet.*	*Ft. In.*
13 0	132.7326	40 10	18 0	254.4696	56 6½	23 0	415.4766	72 3
1	134.4391	41 1¼	1	256 8303	56 9⅝	1	418.4915	72 6¼
2	136.1574	41 4⅜	2	259.2033	57 ⅞	2	421 5192	72 9⅜
3	137.8867	41 7½	3	261.5872	57 4	3	424.5577	73 ½
4	139.626	41 10⅝	4	263.9807	57 7⅛	4	427.6055	73 3⅝
5	141.3771	42 1¾	5	266.3864	57 10¼	5	430.6658	73 6¾
6	143.1391	42 4⅞	6	268.8031	58 1⅜	6	433.7371	73 9⅞
7	144.9111	42 8	7	271.2293	58 4½	7	436.8175	74 1
8	146.6949	42 11¼	8	273.6678	58 7⅝	8	439.9106	74 4¼
9	148.4896	43 2⅜	9	276.1171	58 10¾	9	443.0146	74 7⅜
10	150.2943	43 5½	10	278.5761	58 2	10	446.1278	74 10½
11	152.1109	43 8⅝	11	281.0472	59 5⅛	11	449.2536	75 1⅝
14 0	153.9384	43 11¾	19 0	283.5294	59 8¼	24 0	452.3904	75 4¾
1	155.7758	44 2⅞	1	286.021	59 11⅜	1	455.5362	75 7⅞
2	157.625	44 6	2	288.5249	60 2½	2	458.6948	75 11
3	159.4852	44 9¼	3	291.0397	60 5⅝	3	461.8642	76 2⅛
4	161.3553	45 ⅜	4	293.5641	60 8¾	4	465.0428	76 5¼
5	163.2373	45 3½	5	296.1107	60 11⅞	5	468.2341	76 8½
6	165.1303	45 6⅝	6	298.6483	60 3⅛	6	471.4363	76 11⅝
7	167.0331	45 9¾	7	301.2054	61 6¼	7	474.6476	77 2¾
8	168.9479	46 ⅞	8	303.7747	61 9⅜	8	477.8716	77 5⅞
9	170.8735	46 4	9	306.355	61 ½	9	481.1065	77 9
10	172.8091	46 7¼	10	308.9448	61 3⅝	10	484.3506	78 ⅛
11	174.7565	46 11¼	11	311.5469	62 6¾	11	487.6073	78 3¼
15 0	176.715	47 1½	20 0	314.16	62 9⅞	25 0	490.875	78 6½
1	178.6832	47 4⅝	1	316.7824	62 1⅛	1	494.1516	78 9½
2	180.6634	47 7¾	2	319.4173	63 4¼	2	497.4411	79 ¾
3	182.6545	47 10⅞	3	322.063	63 7⅜	3	500.7415	79 3⅞
4	184.6555	48 2⅛	4	324.7182	63 11½	4	504.051	79 7⅛
5	186.6684	48 5¼	5	327.3858	63 1⅝	5	507.3732	79 11¼
6	188.6923	48 8⅜	6	330.0643	64 4¾	6	510.7063	80 1⅜
7	190.726	48 11½	7	332.7522	64 7⅞	7	514.0484	80 4⅝
8	192.7716	49 2⅝	8	335.4525	64 11	8	517.4034	80 7¾
9	194.8282	49 5¾	9	338.1637	65 2¼	9	520.7692	80 10⅞
10	196.8946	49 8⅞	10	340.8844	65 5⅜	10	524.1441	81 1⅞
11	198.973	50 0	11	343.6174	65 8½	11	527.5318	81 5
16 0	201.0624	50 3¼	21 0	346.3614	65 11⅝	26 0	530.9304	81 8¼
1	203.1615	50 6⅜	1	349.1147	66 2¾	1	534.3379	81 11⅜
2	205.2726	50 9½	2	351.8804	66 5⅞	2	537.7583	82 2½
3	207.3946	51 ½	3	354.6571	66 9	3	541.1896	82 5⅝
4	209.5264	51 3¾	4	357.4432	66 ⅛	4	544.6299	82 8¾
5	211.6703	51 6⅞	5	360.2417	67 3⅜	5	548.083	82 11⅞
6	213.8251	51 10	6	363.0511	67 6½	6	551.5471	83 3
7	215.9896	52 1¼	7	365.8698	67 9⅝	7	555.0201	83 6¼
8	218.1662	52 4⅜	8	368.7011	68 ¾	8	558.5059	83 9⅜
9	220.3537	52 7½	9	371.5432	68 3⅞	9	562.0027	84 ½
10	222.551	52 10½	10	374.3947	68 7	10	565.5084	84 3⅝
11	224.7603	53 1⅝	11	377.2587	68 10¼	11	569.027	84 6¾
17 0	226.9806	53 4⅞	22 0	380.1336	69 1⅜	27 0	572.5566	84 9⅞
1	229.2105	53 8	1	383.0177	69 4½	1	576.0949	85 1
2	231.4525	53 11⅛	2	385.9144	69 7⅝	2	579.6463	85 4¼
3	233.7055	54 2⅜	8	383.822	69 10¾	8	583.2085	85 8⅜
4	235.9682	54 5⅜	4	391.7389	70 1⅞	4	586.7796	85 11½
5	238.243	54 8½	5	394.6683	70 5	5	590.3637	86 1⅝
6	240.5287	54 11⅝	6	397.6087	70 8¼	6	593.9587	86 4¾
7	242.8241	55 2⅞	7	400.5583	70 11⅜	7	597.5625	86 7⅞
8	245.1316	55 6	8	403.5204	71 2½	8	601.1793	86 11
9	247.45	55 9⅛	9	406.4935	71 5⅝	9	604.807	87 2¼
10	249.7781	56 ¼	10	409.4759	71 8¾	10	608.4436	87 5⅜
11	252.1184	56 3½	11	412.4707	71 11⅞	11	612.0931	87 8½

Areas and Circumferences of Circles (Feet and Inches).

Diam.	Area.	Circum.	Diam.	Area.	Circum.	Diam.	Area.	Circum.
Ft.	*Feet.*	*Ft. In.*	*Ft.*	*Feet.*	*Ft. In.*	*Ft.*	*Feet.*	*Ft. In.*
28 0	615.7536	87 11½	33 0	855.301	103 8	38 0	1134.118	119 4½
1	619.4228	88 2⅝	1	859.624	103 11⅛	1	1139.095	119 7⅝
2	623.105	88 5¾	2	863.961	104 2¼	2	1144.087	119 10¾
3	626.7982	88 9	3	868.309	104 5⅜	3	1149.089	120 2
4	630.5002	89 ⅛	4	872.665	104 8⅝	4	1154.110	120 5⅛
5	634.2152	89 3¼	5	877.035	104 11¾	5	1159.124	120 8¼
6	637.9411	89 6⅜	6	881.415	105 2⅞	6	1164.159	120 11⅜
7	641.6758	89 9½	7	885.804	105 6	7	1169.202	121 2½
8	645.4235	90 ⅝	8	890.206	105 9⅛	8	1174.259	121 5⅝
9	649.1821	90 3¾	9	894.619	106 ¼	9	1179.327	121 8¾
10	652.9495	90 6⅞	10	899.041	106 3⅜	10	1184.403	121 11⅞
11	656.73	90 11⅛	11	903.476	106 6⅝	11	1189.493	122 3⅛
29 0	660.5214	91 1¼	34 0	907.922	106 9¾	39 0	1194.593	122 6¼
1	664.3214	91 4⅜	1	912.377	107 ⅞	1	1199.719	122 9½
2	668.1346	91 7½	2	916.844	107 4	2	1204.824	123 ½
3	671.9587	91 10⅝	3	921.323	107 7⅛	3	1209.958	123 3⅝
4	675.7915	92 1¾	4	925.810	107 10¼	4	1215.099	123 6¾
5	679.6375	92 4⅞	5	930.311	108 1⅜	5	1220.254	123 9⅞
6	683.4943	92 8⅛	6	934.822	108 4⅝	6	1225.420	124 1⅛
7	687.3598	92 11¼	7	939.342	108 7¾	7	1230.594	124 4¼
8	691.2385	93 2⅜	8	943.875	108 10⅞	8	1235.782	124 7⅜
9	695.1028	93 5½	9	948.419	109 2	9	1240.981	124 10½
10	699.0263	93 8⅝	10	952.972	109 5⅛	10	1246.188	125 1⅝
11	702.9377	93 11⅞	11	957.538	109 8¼	11	1251.408	125 4¾
30 0	706.86	94 2⅞	35 0	962.115	109 11⅜	40 0	1256.64	125 7⅞
1	710.791	94 6	1	966.770	110 2⅝	1	1261.879	125 11
2	714.735	94 9¼	2	971.299	110 5¾	2	1267.133	126 2¼
3	718.69	95 ⅜	3	975.908	110 8⅞	3	1272.397	126 5⅜
4	722.654	95 3½	4	980.526	111 0	4	1277.669	126 8½
5	726.631	95 6⅝	5	985.158	111 3⅛	5	1282.955	126 11⅝
6	730.618	95 9¾	6	989.803	111 6¼	6	1288.252	127 2¾
7	734.615	96 ⅞	7	994.451	111 9⅜	7	1293.557	127 5⅞
8	738.624	96 4	8	999.115	112 ½	8	1298.876	127 9
9	742.645	96 7¼	9	1003.79	112 3¾	9	1304.206	128 ¼
10	746.674	96 10⅜	10	1008.473	112 6⅞	10	1309.543	128 3⅜
11	750.716	97 1½	11	1013.170	112 10	11	1314.895	128 6½
31 0	754.769	97 4⅝	36 0	1017.878	113 1⅛	41 0	1320.257	128 9⅝
1	758.831	97 7¾	1	1022.594	113 4¼	1	1325.628	129 ¾
2	762.906	97 10⅞	2	1027.324	113 7⅜	2	1331.012	129 3⅞
3	766.992	98 2	3	1032.064	113 10⅝	3	1336.407	129 7
4	771.086	98 5⅛	4	1036.813	114 1¾	4	1341.810	129 10⅛
5	775.191	98 8⅜	5	1041.576	114 4⅞	5	1347.227	130 1⅜
6	779.313	98 11½	6	1046.349	114 8	6	1352.655	130 4½
7	783.440	99 2⅝	7	1051.130	114 11⅛	7	1358.091	130 7⅝
8	787.581	99 5¾	8	1055.926	115 2¼	8	1363.541	130 10¾
9	791.732	99 8⅞	9	1060.731	115 5⅜	9	1369.001	131 1⅞
10	795.892	100 0	10	1065.546	115 9¼	10	1374.47	131 5
11	800.065	100 3⅛	11	1070.374	115 11⅝	11	1379.952	131 8¼
32 0	804.25	100 6⅜	37 0	1075.2126	116 2⅞	42 0	1385.446	131 11⅜
1	808.442	100 9½	1	1080.059	116 6	1	1390.247	132 2½
2	812.648	101 ⅝	2	1084.920	116 9⅛	2	1396.462	132 5⅝
3	816.865	101 3¾	3	1089.791	117 ¼	3	1401.988	132 8¾
4	821.090	101 6⅞	4	1094.671	117 3½	4	1407.522	132 11⅞
5	825.329	101 10	5	1099.564	117 6½	5	1413.07	133 3
6	829.579	102 1⅛	6	1104.469	117 9⅝	6	1418.629	133 6¼
7	833.837	102 4⅜	7	1109.381	118 ¾	7	1424.195	133 9¼
8	838.103	102 7½	8	1114.307	118 4	8	1429.776	134 ½
9	842.391	102 10⅝	9	1119.244	118 7⅛	9	1435.367	134 3⅝
10	846.681	103 1¾	10	1124.189	118 10¼	10	1440.967	134 6¾
11	850.985	103 4⅞	11	1129.148	119 1⅜	11	1446.580	134 9⅞

Areas and Circumferences of Circles (Feet and Inches).

Diam.	Area.	Circum.	Diam.	Area.	Circum.	Diam.	Area.	Circum.
Ft.	*Feet.*	*Ft. In.*	*Ft.*	*Feet.*	*Ft. In.*	*Ft.*	*Feet.*	*Ft. In.*
43 0	1452.205	135 1	46 0	1661.906	144 6⅛	49 0	1885.745	153 11¼
1	1457.836	135 4⅛	1	1667.931	144 9¼	1	1892.172	154 2⅜
2	1463.483	135 7¼	2	1673 97	145 ⅜	2	1898.504	154 5½
3	1469.14	135 10½	3	1680.02	145 3½	3	1905.037	154 8⅝
4	1474.804	136 1⅝	4	1686.077	145 6⅝	4	1911.497	154 11⅞
5	1480.483	136 4¾	5	1692.148	145 9⅞	5	1917.961	155 2⅞
6	1486.173	136 7⅞	6	1698.231	146 1⅛	6	1924.426	155 6
7	1491.870	136 11	7	1704.321	146 4¼	7	1930.919	155 9¼
8	1497.582	137 2⅛	8	1710.425	146 7⅜	8	1937.316	156 ⅜
9	1503.305	137 5¼	9	1716.541	146 10½	9	1943.914	156 3½
10	1509.035	137 8⅜	10	1722.663	147 1½	10	1950.439	156 6⅝
11	1514.779	137 11⅝	11	1728.801	147 4⅝	11	1956.969	156 9¾
44 0	1520.534	138 2¾	47 0	1734.947	147 7¾	50 0	1963.5	157 ⅞
1	1526.297	138 5⅞	1	1741.104	147 11			
2	1532.074	138 9	2	1747.274	148 2⅛			
3	1537.862	139 ⅛	3	1753.455	148 5¼			
4	1543.658	139 3¼	4	1759.643	148 8⅜			
5	1549.478	139 6⅜	5	1765.845	148 11½			
6	1555.288	139 9⅝	6	1772.059	149 2⅝			
7	1561.116	140 ¾	7	1778.28	149 5⅞			
8	1566.959	140 3⅞	8	1784.515	149 8⅞			
9	1572.812	140 7½	9	1790.761	150 ⅛			
10	1578.673	141 10⅛	10	1797.015	150 3¼			
11	1584.549	141 1¼	11	1803.283	150 6⅜			
45 0	1590.435	141 4⅜	48 0	1809.562	150 9½			
1	1596.329	141 7½	1	1815.848	151 ⅝			
2	1602.237	141 10¾	2	1822.149	151 3¾			
3	1608.155	142 1⅞	3	1828.460	151 6⅞			
4	1614.082	142 5	4	1834.779	151 10⅛			
5	1620.023	142 8⅛	5	1841.173	152 1¼			
6	1625.974	142 11¼	6	1847.457	152 4⅜			
7	1631.933	143 2⅜	7	1853.809	152 7½			
8	1637.907	143 5½	8	1860.175	152 10⅝			
9	1643.891	143 8¾	9	1866.552	153 1¾			
10	1649.883	143 11⅞	10	1872.937	153 4⅞			
11	1655.889	144 3	11	1879.335	153 8⅛			

Circular Arcs.

To find the length of a circular arc when its chord and height, or versed sine is given; BY THE FOLLOWING TABLE.

RULE. — Divide the height by the chord; find in the column of heights the number equal to this quotient. Take out the corresponding number from the column of lengths. Multiply this number by the given chord.

EXAMPLE. — The chord of an arc is 80 and its versed sine is 30, what is the length of the arc?

Ans. $30 \div 80 = 0.375$. The length of an arc for a height of 0.375 we find from table to be 1.34063. $80 \times 1.34063 = 107.2504 =$ length of arc.

TABLE OF CIRCULAR ARCS.

Hghts.	Lengths.	Hghts.	Lengths.	Hghts.	Lengths.	Hghts.	Lengths.	Hghts.	Lengths.
.001	1.00001	.062	1.01021	.123	1.03987	.184	1.08797	.245	1.15308
.002	1.00001	.063	1.01054	.124	1.04051	.185	1.08890	.246	1.15428
.003	1.00002	.064	1.01088	.125	1.04116	.186	1.08984	.247	1.15549
.004	1.00004	.065	1.01123	.126	1.04181	.187	1.09079	.248	1.15670
.005	1.00007	.066	1.01158	.127	1.04247	.188	1.09174	.249	1.15791
.006	1.00010	.067	1.01193	.128	1.04313	.189	1.09269	.250	1.15912
.007	1.00013	.068	1.01228	.129	1.04380	.190	1.09365	.251	1.16034
.008	1.00017	.069	1.01264	.130	1.04447	.191	1.09461	.252	1.16156
.009	1.00022	.070	1.01301	.131	1.04515	.192	1.09557	.253	1.16279
.010	1.00027	.071	1.01338	.132	1.04584	.193	1.09654	.254	1.16402
.011	1.00032	.072	1.01376	.133	1.04652	.194	1.09752	.255	1.16526
.012	1.00038	.073	1.01414	.134	1.04722	.195	1.09850	.256	1.16650
.013	1.00045	.074	1.01453	.135	1.04792	.196	1.09949	.257	1.16774
.014	1.00053	.075	1.01493	.136	1.04862	.197	1.10048	.258	1.16899
.015	1.00061	.076	1.01533	.137	1.04932	.198	1.10147	.259	1.17024
.016	1.00069	.077	1.01573	.138	1.05003	.199	1.10247	.260	1.17150
.017	1.00078	.078	1.01614	.139	1.05075	.200	1.10347	.261	1.17276
.018	1.00087	.079	1.01656	.140	1.05147	.201	1.10447	.262	1.17403
.019	1.00097	.080	1.01698	.141	1.05220	.202	1.10548	.263	1.17530
.020	1.00107	.081	1.01741	.142	1.05293	.203	1.10650	.264	1.17657
.021	1.00117	.082	1.01784	.143	1.05367	.204	1.10752	.265	1.17784
.022	1.00128	.083	1.01828	.144	1.05441	.205	1.10855	.266	1.17912
.023	1.00140	.084	1.01872	.145	1.05516	.206	1.10958	.267	1.18040
.024	1.00153	.085	1.01916	.146	1.05591	.207	1.11062	.268	1.18169
.025	1.00167	.086	1.01961	.147	1.05667	.208	1.11165	.269	1.18299
.026	1.00182	.087	1.02006	.148	1.05743	.209	1.11269	.270	1.18429
.027	1.00196	.088	1.02052	.149	1.05819	.210	1.11374	.271	1.18559
.028	1.00210	.089	1.02098	.150	1.05896	.211	1.11479	.272	1.18689
.029	1.00225	.090	1.02145	.151	1.05973	.212	1.11584	.273	1.18820
.030	1.00240	.091	1.02192	.152	1.06051	.213	1.11690	.274	1.18951
.031	1.00256	.092	1.02240	.153	1.06130	.214	1.11796	.275	1.19082
.032	1.00272	.093	1.02289	.154	1.06209	.215	1.11904	.276	1.19214
.033	1.00289	.094	1.02339	.155	1.06288	.216	1.12011	.277	1.19346
.034	1.00307	.095	1.02389	.156	1.06368	.217	1.12118	.278	1.19479
.035	1.00327	.096	1.02440	.157	1.06449	.218	1.12225	.279	1.19612
.036	1.00345	.097	1.02491	.158	1.06530	.219	1.12334	.280	1.19746
.037	1.00364	.098	1.02542	.159	1.06611	.220	1.12444	.281	1.19880
.038	1.00384	.099	1.02593	.160	1.06693	.221	1.12554	.282	1.20014
.039	1.00405	.100	1.02645	.161	1.06775	.222	1.12664	.283	1.20149
.040	1.00426	.101	1.02698	.162	1.06858	.223	1.12774	.284	1.20284
.041	1.00447	.102	1.02752	.163	1.06941	.224	1.12885	.285	1.20419
.042	1.00469	.103	1.02806	.164	1.07025	.225	1.12997	.286	1.20555
.043	1.00492	.104	1.02860	.165	1.07109	.226	1.13108	.287	1.20691
.044	1.00515	.105	1.02914	.166	1.07194	.227	1.13219	.288	1.20827
.045	1.00539	.106	1.02970	.167	1.07279	.228	1.13331	.289	1.20964
.046	1.00563	.107	1.03026	.168	1.07365	.229	1.13444	.290	1.21102
.047	1.00587	.108	1.03082	.169	1.07451	.230	1.13557	.291	1.21239
.048	1.00612	.109	1.03139	.170	1.07537	.231	1.13671	.292	1.21377
.049	1.00638	.110	1.03196	.171	1.07624	.232	1.13785	.293	1.21515
.050	1.00665	.111	1.03254	.172	1.07711	.233	1.13900	.294	1.21654
.051	1.00692	.112	1.03312	.173	1.07799	.234	1.14015	.295	1.21794
.052	1.00720	.113	1.03371	.174	1.07888	.235	1.14131	.296	1.21933
.053	1.00748	.114	1.03430	.175	1.07977	.236	1.14247	.297	1.22073
.054	1.00776	.115	1.03490	.176	1.08066	.237	1.14363	.298	1.22213
.055	1.00805	.116	1.03551	.177	1.08156	.238	1.14480	.299	1.22354
.056	1.00834	.117	1.03611	.178	1.08246	.239	1.14597	.300	1.22495
.057	1.00864	.118	1.03672	.179	1.08337	.240	1.14714	.301	1.22636
.058	1.00895	.119	1.03734	.180	1.08428	.241	1.14832	.302	1.22778
.059	1.00926	.120	1.03797	.181	1.08519	.242	1.14951	.303	1.22920
.060	1.00957	.121	1.03860	.182	1.08611	.243	1.15070	.304	1.23063
.061	1.00989	.122	1.03923	.183	1.08704	.244	1.15189	.305	1.23206

Table of Circular Arcs (*concluded*).

Hghts.	Lengths.	Hghts.	Lengths.	Hghts.	Lengths.	Hghts.	Lengths.	Hghts.	Lengths.
.306	1.23349	.345	1.29209	.384	1.35575	.423	1.42402	.462	1.49651
.307	1.23492	.346	1.29366	.385	1.35744	.424	1.42583	.463	1.49842
.308	1.23636	.347	1.29523	.386	1.35914	.425	1.42764	.464	1.50033
.309	1.23781	.348	1.29681	.387	1.36084	.426	1.42945	.465	1.50224
.310	1.23926	.349	1.29839	.388	1.36254	.427	1.43127	.466	1.50416
.311	1.24070	.350	1.29997	.389	1.36425	.428	1.43309	.467	1.50608
.312	1.24216	.351	1.30156	.390	1.36596	.429	1.43491	.468	1.50800
.313	1.24361	.352	1.30315	.391	1.36767	.430	1.43673	.469	1.50992
.314	1.24507	.353	1.30474	.392	1.36939	.431	1.43856	.470	1.51185
.315	1.24654	.354	1.30634	.393	1.37111	.432	1.44039	.471	1.51378
.316	1.24801	.355	1.30794	.394	1.37283	.433	1.44222	.472	1.51571
.317	1.24948	.356	1.30954	.395	1.37455	.434	1.44405	.473	1.51764
.318	1.25095	.357	1.31115	.396	1.37628	.435	1.44589	.474	1.51958
.319	1.25243	.358	1.31276	.397	1.37801	.436	1.44773	.475	1.52152
.320	1.25391	.359	1.31437	.398	1.37974	.437	1.44957	.476	1.52346
.321	1.25540	.360	1.31599	.399	1.38148	.438	1.45142	.477	1.52541
.322	1.25689	.361	1.31761	.400	1.38322	.439	1.45327	.478	1.52736
.323	1.25838	.362	1.31923	.401	1.38496	.440	1.45512	.479	1.52931
.324	1.25988	.363	1.32086	.402	1.38671	.441	1.45697	.480	1.53126
.325	1.26138	.364	1.32249	.403	1.38846	.442	1.45883	.481	1.53322
.326	1.26288	.365	1.32413	.404	1.39021	.443	1.46069	.482	1.53518
.327	1.26437	.366	1.32577	.405	1.39196	.444	1.46255	.483	1.53714
.328	1.23588	.367	1.32741	.406	1.39372	.445	1.46441	.484	1.53910
.329	1.23740	.368	1.32905	.407	1.39548	.446	1.46628	.485	1.54106
.330	1.26892	.369	1.33069	.408	1.39724	.447	1.46815	.486	1.54302
.331	1.27044	.370	1.33234	.409	1.39900	.448	1.47002	.487	1.54499
.332	1.27195	.371	1.33399	.410	1.40077	.449	1.47189	.488	1.54696
.333	1.27349	.372	1.33564	.411	1.40254	.450	1.47377	.489	1.54893
.334	1.27502	.373	1.33730	.412	1.40432	.451	1.47565	.490	1.55091
.335	1.27656	.374	1.33896	.413	1.40610	.452	1.47753	.491	1.55289
.336	1.27810	.375	1.34063	.414	1.40788	.453	1.47942	.492	1.55487
.337	1.27964	.376	1.34229	.415	1.40966	.454	1.48131	.493	1.55685
.338	1.28118	.377	1.34396	.416	1.41145	.455	1.48320	.494	1.55884
.339	1.28273	.378	1.34563	.417	1.41324	.456	1.48509	.495	1.56083
.340	1.28428	.379	1.34731	.418	1.41503	.457	1.48699	.496	1.56282
.341	1.28583	.380	1.34899	.419	1.41682	.458	1.48889	.497	1.56481
.342	1.28739	.381	1.35068	.420	1.41861	.459	1.49079	.498	1.56681
.343	1.28895	.382	1.35237	.421	1.42041	.460	1.49269	.499	1.56881
.344	1.29052	.383	1.35406	.422	1.42221	.461	1.49460	.500	1.57080

Table of Lengths of Circular Arcs whose Radius is 1.

RULE.—Knowing the measure of the circle and the measure of the arc in degrees, minutes, and seconds; take from the table the lengths opposite the number of degrees, minutes, and seconds in the arc, and multiply their sum by the radius of the circle.

EXAMPLE.—What is the length of an arc subtending an angle of 13° 27′ 8″, with a radius of 8 feet.

$$
\begin{array}{rr}
\textit{Ans.}\ \text{Length for } 13^\circ = & 0.2268928 \\
27' = & 0.0078540 \\
8'' = & \underline{0.0000388} \\
13^\circ\,27'\,8'' = & 0.2347856 \\
 & \underline{8} \\
\text{Length of arc} = & 1.8782848 \text{ feet.}
\end{array}
$$

Lengths of Circular Arcs; Radius = 1.

To compute the chord of an arc when the chord of half the arc and the versed sine are given. (The versed sine is the perpendicular bo, Fig. 31.)

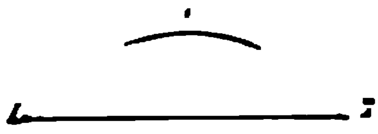


Fig. 31

RULE. — From the square of the chord of half the arc subtract the square of the versed sine, and take twice the square root of the remainder.

EXAMPLE. — The chord of half the arc is 60, and the versed sine 36; what is the length of the chord of the arc?

Ans. $60^2 - 36^2 = 2304$, and $\sqrt{2304} = 48$, and $48 \times 2 = 96$, the chord.

To compute the chord of an arc when the diameter and versed sine are given.

RULE. — Multiply the versed sine by 2, and subtract the product from the diameter; then subtract the square of the remainder from the square of the diameter, and take the square root of that remainder.

EXAMPLE. — The diameter of a circle is 100, and the versed sine of an arc 36; what is the chord of the arc?

Ans. $36 \times 2 = 72$. $100 - 72 = 28$. $100^2 - 28^2 = 9216$. $\sqrt{9216} = 96$, the chord of the arc.

To compute the chord of half an arc when the chord of the arc and its versed sine are given.

RULE. — Take the square root of the sum of the squares of the versed sine and of half the chord of the arc.

EXAMPLE. — The chord of an arc is 96, and the versed sine 36, what is the chord of half the arc?

Ans. $\sqrt{36^2 + 48^2} = 60$.

To compute the chord of half an arc when the diameter and versed sine are given.

RULE. — Multiply the diameter by the versed sine, and take the square root of their product.

To compute the diameter.

RULE 1. — Divide the square of the chord of half the arc by the versed sine.

RULE 2. — Add the square of half the chord of the arc to the square of the versed sine, and divide this sum by the versed sine.

EXAMPLE.—What is the radius of an arc whose chord is 96, and whose versed sine is 36?

Ans. $48^2 + 36^2 = 3600$. $3600 \div 36 = 100$, the diameter, and radius $= 50$.

To compute the versed sine.

RULE.—Divide the square of the chord of half the arc by the diameter.

To compute the versed sine when the chord of the arc and the diameter are given.

RULE.—From the square of the diameter subtract the square of the chord, and extract the square root of the remainder; subtract this root from the diameter, and halve the remainder.

To compute the length of an arc of a circle when the number of degrees and the radius are given.

RULE 1.—Multiply the number of degrees in the arc by 3.1416 multiplied by the radius, and divide by 180. The result will be the length of the arc in the same unit as the radius.

RULE 2.—Multiply the radius of the circle by 0.01745, and the product by the degrees in the arc.

EXAMPLE.—The number of degrees in an arc is 60, and the radius is 10 inches, what is the length of the arc in inches?

Ans. $10 \times 3.1416 \times 60 = 1884.96 \div 180 = 10.47$ inches; or, $10 \times 0.01745 \times 60 = 10.47$ inches.

To compute the length of the arc of a circle when the length is given in degrees, minutes, and seconds.

RULE 1.—Multiply the number of degrees by 0.01745329, and the product by the radius.

RULE 2.—Multiply the number of minutes by 0.00029, and that product by the radius.

RULE 3.—Multiply the number of seconds by 0.00000448 times the radius. Add together these three results for the length of the arc.

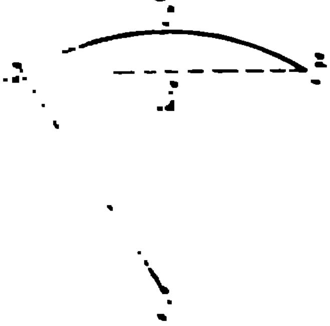
See also table, p. 57.

EXAMPLE.—What is the length of an arc of 60° 10′ 5″, the radius being 4 feet?

$$
\begin{array}{llll}
\textit{Ans.}\ 1. & 60^\circ \times 0.01745329 & \times 4 = 4.188789 & \text{feet.} \\
2. & 10' \times 0.00029 & \times 4 = 0.0116 & \text{feet.} \\
3. & 5'' \times 0.0000048 & \times 4 = 0.000096 & \text{feet.} \\
\hline
 & & 4.200485 & \text{feet.}
\end{array}
$$

[illegible] *a sector of [illegible] circle when the degrees of the [illegible] and the radius are given* (Fig. 32).

Fig. 32



RULE. — Multiply the number of degrees in the [illegible] by the area of the whole circle, and divide by 360.

EXAMPLE. — What is the area of a sector of a circle whose radius is 5, and the length of the arc is [illegible]

[illegible] Area of circle $= 10 \times 10 \times 0.7854 = 78.54$.

Then area of sector $= \dfrac{78.5 \times 60}{360} = 13.09$.

[illegible] *degrees and minutes*, reduce it [illegible] and multiply by the area of the whole circle, and divide [illegible]

[illegible] *a circle when the length of the* [illegible]

[illegible] — [illegible] the length of the arc by half the length of the [illegible] area.

[illegible] *circle when the chord and* [illegible] *or diameter of the circle* [illegible]

[illegible] Fig. 32).

[illegible] *a semicircle*). — Ascer[illegible] having the same arc as the segment, [illegible] triangle formed by the chord of the [illegible] the sector, and take the difference of [illegible]

[illegible] *than a semicircle*). — As[illegible] the area of the lesser portion of the [illegible] area of the whole circle, and the remain[illegible]

[illegible]

[illegible] by the circumference, and the [illegible]

[illegible] surface of a sphere of 10 inches [illegible]

[illegible] 3.1416 = 31.416 inches; [illegible] the surface of sphere.

To compute the surface of a segment of a sphere.

RULE.—Multiply the height (bc, Fig. 33) by the circumference of the sphere, and add the product to the area of the base.

To find the area of the base, we have the diameter of the sphere and the length of the versed sine of the arc abd, and we can find the length of the chord ad by the rule on p. 56. Having, then, the length of the chord ad for the diameter of the base, we can easily find the area.

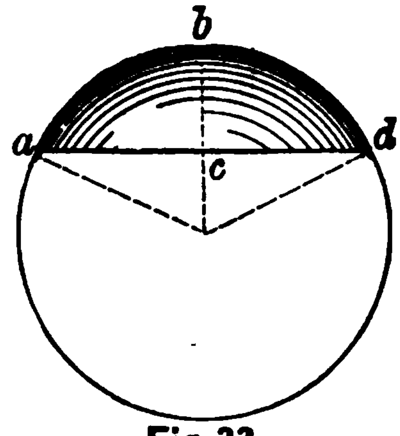


Fig. 33

EXAMPLE.—The height, bc, of a segment abd, is 36 inches, and the diameter of the sphere is 100 inches. What is the convex surface, and what the whole surface?

Ans. $100 \times 3.1416 = 314.16$ inches, the circumference of sphere.

$36 \times 314.16 = 11309.76$, the convex surface.

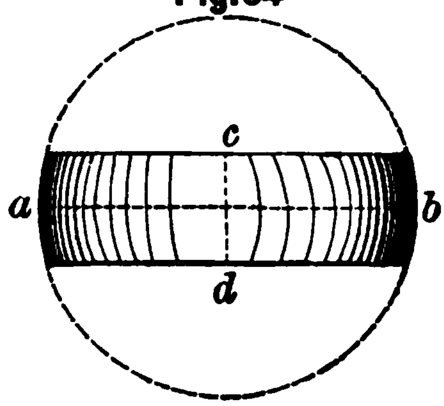
The length of $ad = 100 - \overline{36 \times 2} = 28$.

$\sqrt{100^2 - 28^2} = 96$, the chord ad.

$96^2 \times 0.7854 = 7238.2464$, the area of base.

$11309.76 + 7238.2464 = 18548.0064$, the total area.

Fig. 34



To compute the surface of a spherical zone.

RULE.—Multiply the height (cd, Fig. 34) by the circumference of the sphere for the convex surface, and add to it the area of the two ends for the whole area.

Spheroids, or Ellipsoids.

DEFINITION.—Spheroids, or ellipsoids, are figures generated by the revolution of a semi-ellipse about one of its diameters.

When the revolution is about the short diameter, they are *prolate*; and, when it is about the long diameter, they are *oblate*.

To compute the surface of a spheroid when the spheroid is prolate.

RULE.—Square the diameters, and multiply the square root of half their sum by 3.1416, and this product by the short diameter.

EXAMPLE.—A prolate spheroid has diameters of 10 and 14 inches, what is its surface?

Ans. $10^2 = 100$, and $14^2 = 196$.

Their sum $= 296$, and $\sqrt{\frac{296}{2}} = 12.1655$.

$12.1655 \times 3.1416 \times 10 = 382.191$ square inches.

To compute the surface of a spheroid when the spheroid is oblate.

RULE.—Square the diameters, and multiply the square root of half their sum by 3.1416, and this product by the long diameter.

To compute the surface of a cylinder.

RULE.—Multiply the length by the circumference for the convex surface, and add to the product the area of the two ends for the whole surface.

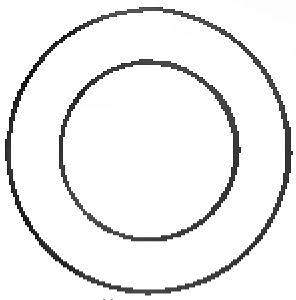


Fig. 35

To compute the sectional area of a circular ring (Fig. 35).

RULE.—Find the area of both circles, and subtract the area of the smaller from the area of the larger: the remainder will be the area of the ring.

To compute the surface of a cone.

RULE.—Multiply the perimeter or circumference of the base by one-half the slant height, or side of the cone, for the convex area. Add to this the area of the base, for the whole area.

EXAMPLE.—The diameter of the base of a cone is 3 inches, and the slant height 15 inches, what is the area of the cone?

Ans. $3 \times 3.1416 =$ 9.4248 = circumference of base.
$9.4248 \times 7\frac{1}{2} =$ 70.686 square inches, the convex surface.
$3 \times 3 \times 0.7854 =$ 7.068 square inches, the area of base.

Area of cone = 77 754 square inches.

Fig. 36

To compute the area of the surface of the frustum of a cone.

RULE.—Multiply the sum of the perimeters of the two ends by the slant height of the frustum, and divide by 2, for the convex surface. Add the area of the top and bottom surfaces.

To compute the surface of a pyramid.

RULE.—Multiply the perimeter of the base by one-half the slant height, and add to the product the area of the base.

To compute the surface of the frustum of a pyramid.

RULE.—Multiply the sum of the perimeters of the two ends by the slant height of the frustum, halve the product, and add to the result the area of the two ends.

MENSURATION OF SOLIDS.

To compute the volume of a prism.

RULE.—Multiply the area of the base by the height.

This rule applies to any prism of any shape on the base, as long as the top and bottom surfaces are parallel.

To compute the volume of a prismoid.

DEFINITION.—A prismoid is a solid having parallel ends or bases dissimilar in shape with quadrilateral sides.

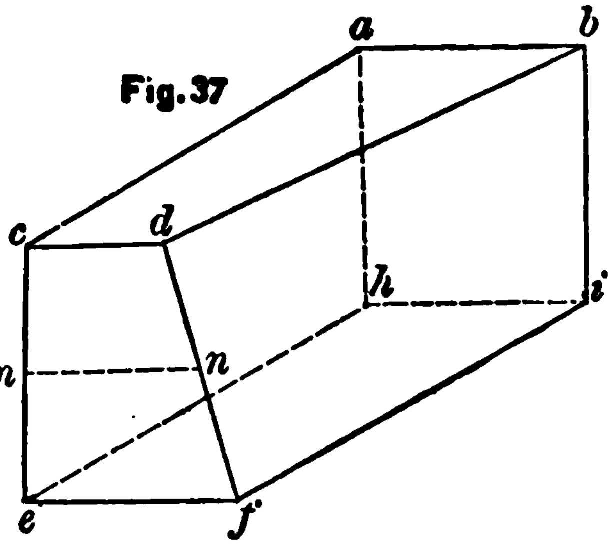


Fig. 37

RULE.—To the sum of the areas of the two ends add four times the area of the middle section parallel to them, and multiply this sum by one-sixth of the perpendicular height.

EXAMPLE.—What is the volume of a quadrangular prismoid, as in Fig. 37, in which $ab = 6''$, $cd = 4''$, $ac = he = 10''$, $ce = 8''$, $ef = 8''$, and $ih = 6''$?

Ans. Area of top $= \frac{6+4}{2} \times 10 = 50.$

Area of bottom $= \frac{8+6}{2} \times 10 = 70.$

Area of middle section $= \frac{6+6}{2} \times 10 = 60.$

$[50 + 70 + (4 \times 60)] \times \frac{10}{6} = 600$ cubic inches.

NOTE.—The length of the end of the middle section, as mn in Fig. 37 $= \frac{cd + ef}{2}$.

To find the volume of a prism truncated obliquely.

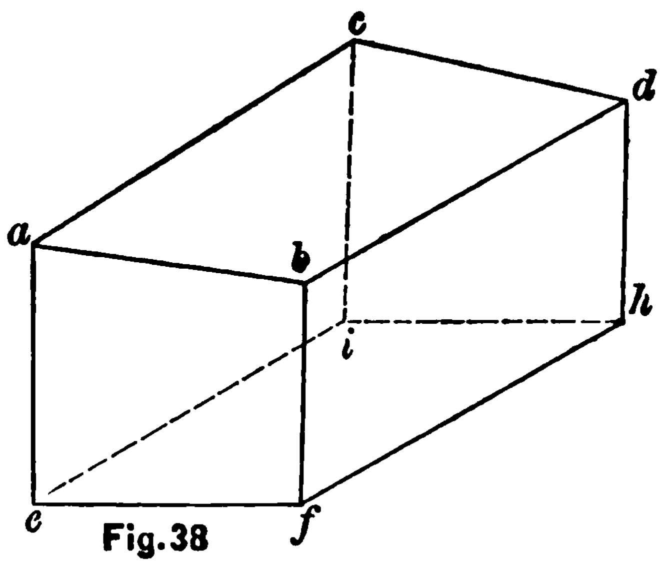


Fig. 38

RULE.—Multiply the area of the base by the average height of the edges.

EXAMPLE.—What is the volume of a truncated prism, as in Fig. 38, where $ef = 6$ inches, $fh = 10$ inches, $ea = 10$, $ci = 12$, $dh = 8$, and $fb = 8$?

Ans. Area of base $= 6 \times 10 = 60$ square inches.

Average height of edges $= \frac{10 + 12 + 8 + 8}{4} = 9\frac{1}{2}$ inches.

$60 \times 9\frac{1}{2} = 970$ cubic inches.

the square of the radius of the base plus the square of the height.

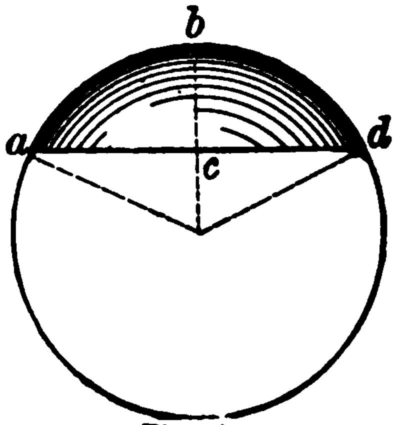


Fig. 41.

$163 \times 4 \times 0.5236 = 341.3872$ cubic inches volume.

SECOND SOLUTION.—By the rule for finding the diameter of a circle when a chord and its versed sine are given, we find that the diameter of the sphere in this case is 16.25 inches; then, by Rule 2, $(3 \times 16.25) - (2 \times 4) = 40.75$, and $40.75 \times 4^2 \times 0.5236 = 341.3872$ cubic inches, the volume of the segment.

To compute the volume of a spherical zone.

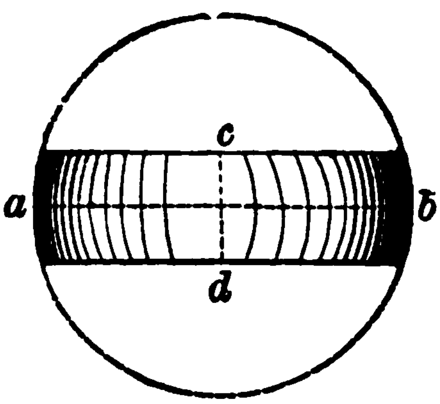


Fig. 42.

DEFINITION.—The part of a sphere included between two parallel planes (Fig. 42).

RULE.—To the sum of the squares of the radii of the two ends add one-third of the square of the height of the zone; multiply this sum by the height, and that product by 1.5708.

To compute the volume of a spheroid.

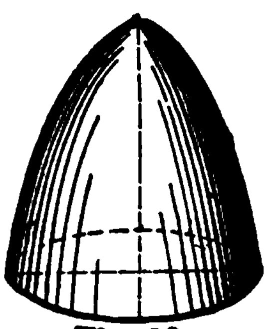


Fig.43

RULE.—Multiply the square of the revolving axis by the fixed axis, and this product by 0.5236.

To compute the volume of a paraboloid of revolution (Fig. 43).

RULE.—Multiply the area of the base by half the altitude.

To compute the volume of a hyperboloid of revolution (Fig. 44).

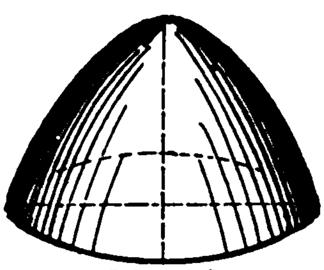


Fig.44

RULE.—To the square of the radius of the base add the square of the middle diameter; multiply this sum by the height, and the product by 0.5236.

To compute the volume of any figure of revolution.

RULE.—Multiply the area of the generating surface by the circumference described by its centre of gravity.

To compute the volume of an excavation, where the ground is irregular, and the bottom of the excavation is level (Fig. 45).

RULE.—Divide the surface of the ground to be excavated into equal squares of about 10 feet on a side, and ascertain by means

of a level the height of each corner, a, a, a, b, b, b, etc., above the level to which the ground is to be excavated. Then add together the heights of all the corners that only come into one square. Next take twice the sum of the heights of all the corners that come in two squares, as b, b, b; next three times the sum of the heights of all the corners that come in three squares, as c, c, c; and then four times the sum of the heights of all the corners that belong to four squares, as d, d, d, etc. Add together all these quantities, and multiply their sum by one-fourth the area of one of the squares. The result will be the volume of the excavation.

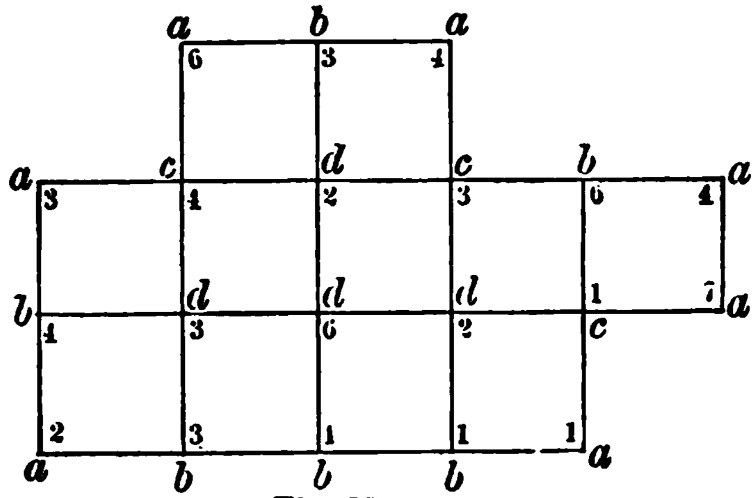


Fig.45

Example. — Let the plan of the excavation for a cellar be as in the figure, and the heights of each corner above the proposed bottom of the cellar be as given by the numbers in the figure, then the volume of the cellar would be as follows, the area of each square being $10 \times 10 = 100$ square feet: —

$$\text{Volume} = \tfrac{1}{4} \text{ of } 100\,(a\text{'s} + 2\,b\text{'s} + 3\,c\text{'s} + 4\,d\text{'s}).$$

$$\begin{aligned}
&\text{The } a\text{'s in this case} &&= 4 + 6 + 3 + 2 + 1 + 7 + 4 &&= 27\\
&2 \times \text{the sum of the } b\text{'s} &&= 2 \times (3 + 6 + 1 + 4 + 3 + 4) &&= 42\\
&3 \times \text{the sum of the } c\text{'s} &&= 3 \times (1 + 3 + 4) &&= 24\\
&4 \times \text{the sum of the } d\text{'s} &&= 4 \times (2 + 3 + 6 + 2) &&= \underline{52}\\
&&&&&\ 145
\end{aligned}$$

Volume $= 25 \times 145 = 3625$ cubic feet, the quantity of earth to be excavated.

GEOMETRICAL PROBLEMS.

PROBLEM 1. — *To bisect, or divide into equal parts, a given line, ab* (Fig. 46).

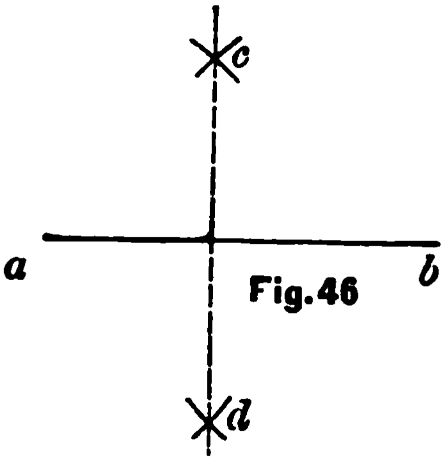


Fig. 46

From *a* and *b*, with any radius greater than half of *ab*, describe arcs intersecting in *c* and *d*. The line *cd*, connecting these intersections, will bisect *ab*, and be perpendicular to it.

PROBLEM 2. — *To draw a perpendicular to a given straight line from a point without it.*

1ST METHOD (Fig. 47). — From the point *a* describe an arc with sufficient radius that it will cut the line *bc* in two places, as *e* and *f*. From *e* and *f* describe two arcs, with the same radius, intersecting in *g*; then a line drawn from *a* to *g* will be perpendicular to the line *bc*.

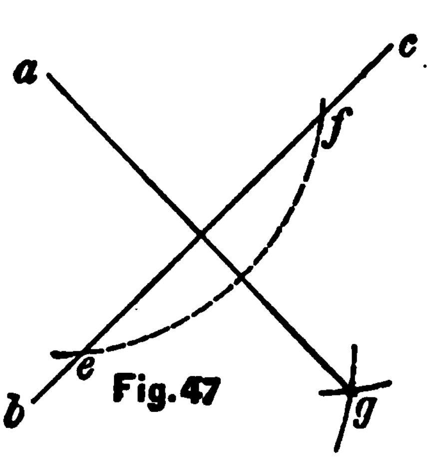


Fig. 47

2D METHOD (Fig. 48). — From any two points, *d* and *c*, at some distance apart in the given line, and with radii *da* and *ca* respectively, describe arcs cutting at *a* and *e*. Draw *ae*, and it will be the perpendicular required. This method is useful where the given point is opposite the end of the line, or nearly so.

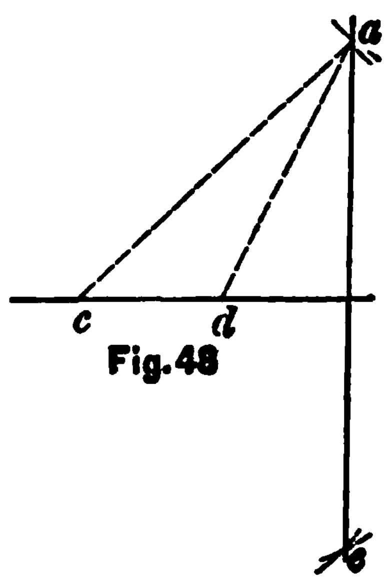


Fig. 48

PROBLEM 3. — *To draw a perpendicular to a straight line from a given point, a, in that line.*

1ST METHOD (Fig. 49). — With any radius, from the given point *a* in the line, describe arcs cutting the line in the points *b* and *c*. Then with *b* and *c* as centres, and with any radius greater than *ab* or *ac*, describe arcs cutting each other at *d*. The line *da* will be the perpendicular desired.

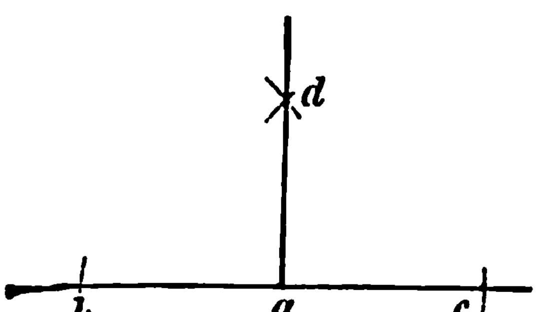


Fig. 49

2D METHOD (Fig. 50, when the given point is at the end of the line). — From any point, b, outside of the line, and with a radius ba, describe a semicircle passing through a, and cutting the given line at d. Through b and d draw a straight line intersecting the semicircle at e. The line ea will then be perpendicular to the line ac at the point a.

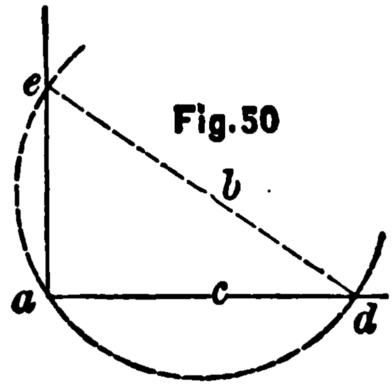


Fig. 50

3D METHOD (Fig. 51) OR THE 3, 4, AND 5 METHOD. — From the point a on the given line measure off 4 inches, or 4 feet, or 4 of any other unit, and with the same unit of measure describe an arc, with a as a centre and 3 units as a radius. Then from b describe an arc, with a radius of 5 units, cutting the first arc in c. Then ca will be the perpendicular. This method is particularly useful in laying out a right angle on the ground, or framing a house where the foot is used as the unit, and the lines laid off by straight edges.

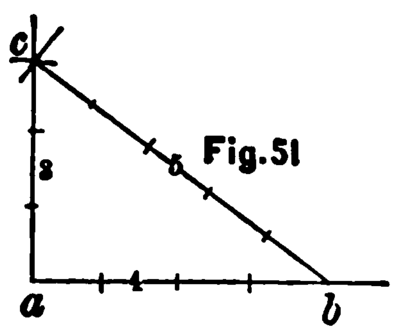


Fig. 51

In laying out a right angle on the ground, the proportions of the triangle may be 30, 40, and 50, or any other multiple of 3, 4, and 5; and it can best be laid out with the tape. Thus, first measure off, say 40 feet from a on the given line, then let one person hold the end of the tape at b, another hold the tape at the 80-foot mark at a, and a third person take hold of the tape at the 50-foot mark, with his thumb and finger, and pull the tape taut. The 50-foot mark will then be at the point c in the line of the perpendicular.

PROBLEM 4. — *To draw a straight line parallel to a given line at a given distance apart* (Fig. 52).

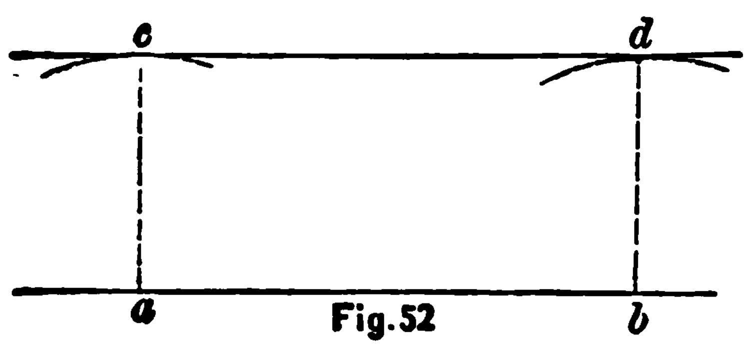


Fig. 52

From any two points near the ends of the given line describe two arcs about opposite the line. Draw the line cd tangent to these arcs, and it will be parallel to ab.

PROBLEM 5. — *To construct an angle equal to a given angle.*

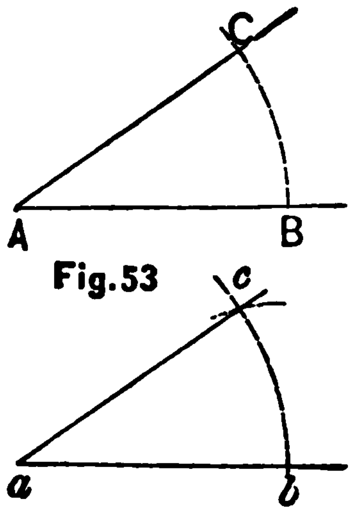


Fig. 53

With the point A, at the apex of the given angle, as a centre, and any radius, describe the arc BC. Then with the point a, at the vertex of the new angle, as a centre, and with the same radius as before, describe an arc like BC. Then with BC as a radius, and b as a centre, describe an arc cutting the other at c. Then will cab be equal to the given angle CAB.

PROBLEM 6. — *From a point on a given line to draw a line making an angle of 60° with the given line* (Fig. 54).

Take any distance, as ab, as a radius, and, with a as a centre, describe the arc bc. Then with b as a centre, and the same radius, describe an arc cutting the first one at c. Draw from a a line through c, and it will make with ab an angle of 60°.

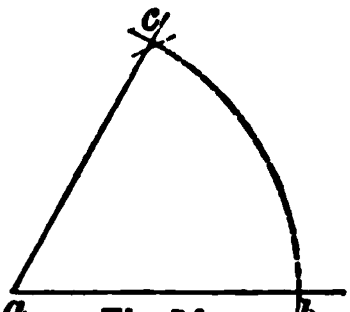


Fig. 54

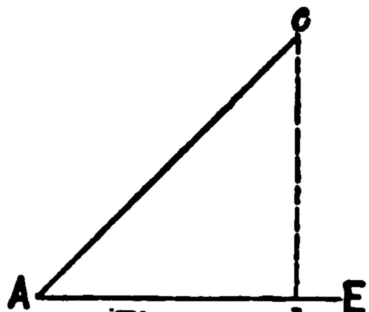


Fig. 55

PROBLEM 7. — *From a given point, A, on a given line, AE, to draw a line making an angle of 45° with the given line* (Fig. 55).

Measure off from A, on AE, any distance, Ab, and at b draw a line perpendicular to AE. Measure off on this perpendicular bc equal to Ab, and draw a line from A through c, and it will make an angle with AE of 45°.

PROBLEM 8. — *From any point, A, on a given line, to draw a line which shall make any desired angle with the given line* (Fig. 56).

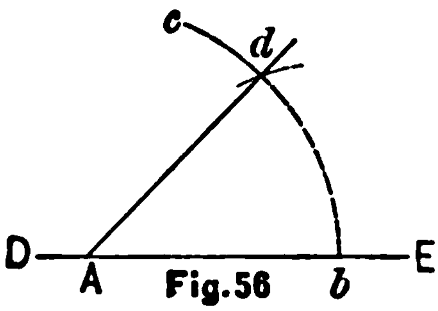


Fig. 56

To perform this problem we must have a table of chords at hand (such as is found on pp. 85–93), which we use as follows. Find in the table the length of chord to a radius 1, for the given angle. Then take any radius, as large as convenient, describe an arc of a circle bc with A as a centre. Multiply the chord of the angle, found in the table, by the length of the radius Ab, and with the product as a new radius, and b as a centre, describe a short arc cutting bc in d. Draw a line from A through d, and it will make the desired angle with DE.

EXAMPLE. — Draw a line from *A* on *DE*, making an angle of 44° 40′ with *DE*.

SOLUTION. — We find that the largest convenient radius for our arc is 8 inches: so with *A* as a centre, and 8 inches as a radius, we describe the arc *bc*. Then, looking in the table of chords, we find the chord for an angle or arc of 44° 40′ to a radius 1 is 0.76. Multiplying this by 8 inches, we have, for the length of our new radius, 6.08 inches, and with this as a radius, and *b* as a centre, we describe an arc cutting *bc* in *d*. *Ad* will then be the line desired.

PROBLEM 9. — *To bisect a given angle, as BAC* (Fig. 57).

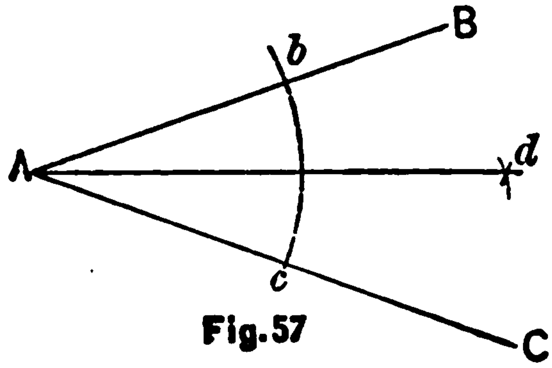


Fig. 57

With *A* as a centre, and any radius, describe an arc, as *cb*. With *c* and *b* as centres, and any radius greater than one-half of *cb*, describe two arcs intersecting in *d*. Draw from *A* a line through *d*, and it will bisect the angle *BAC*.

PROBLEM 10. — *To bisect the angle contained between two lines, as AB and CD, when the vertex of the angle is not on the drawing* (Fig. 58)

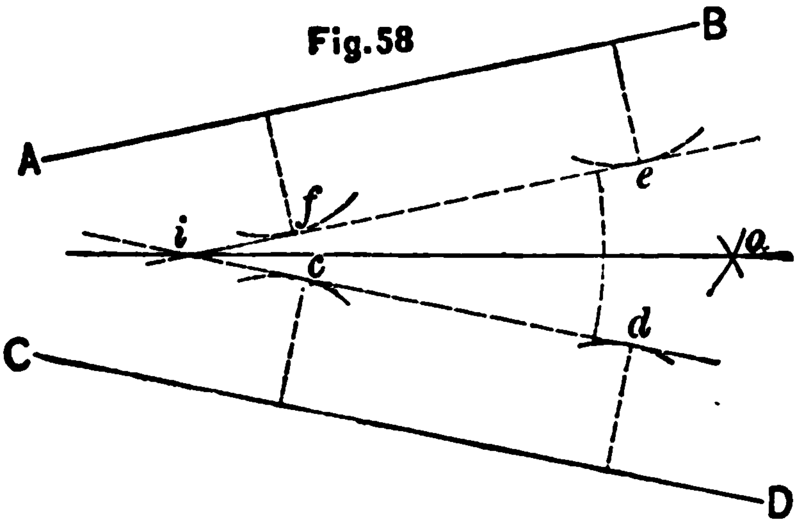


Fig. 58

Draw *fe* parallel to *AB*, and *cd* parallel to *CD*, so that the two lines will intersect each other, as at *i*. Bisect the angle *cid*, as in the preceding problem, and draw a line through *i* and *o* which will bisect the angle between the two given lines.

PROBLEM 11. — *Through two given points, B and C, to describe an arc of a circle with a given radius* (Fig. 59).

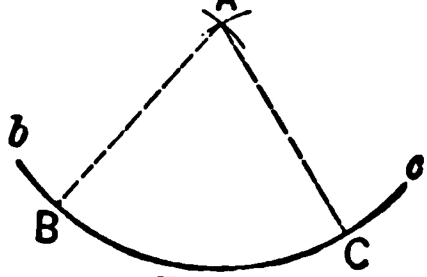


Fig. 59

With *B* and *C* as centres, and a radius equal to the given radius, describe two arcs intersecting at *A*. With *A* as a centre, and the same radius, describe the arc *bc*, which will be found to pass through the given points, *B* and *C*.

PROBLEM 12. — *To find the centre of a given circle* (Fig. 60).

Draw any chord in the circle, as *ab*, and bisect this chord by the perpendicular *cd*. This line will pass through the centre of the circle, and *ef* will be a diameter of the circle. Bisect *ef*, and the centre *o* will be the centre of the circle.

Fig. 60

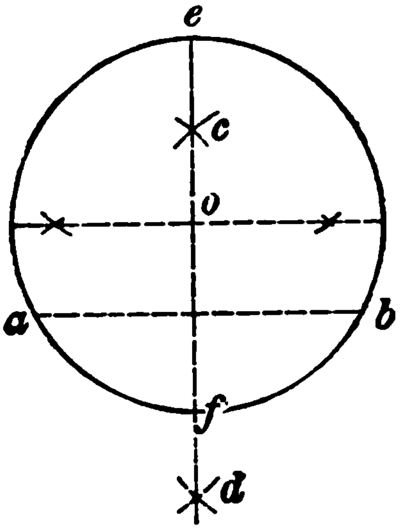
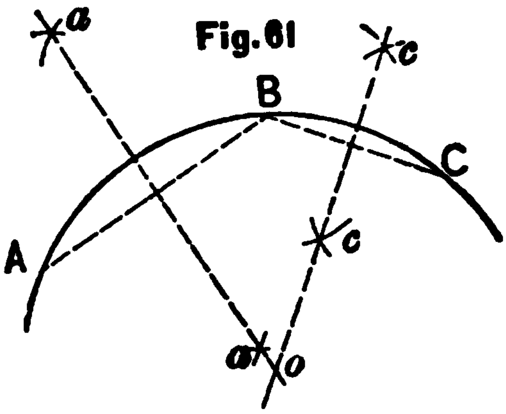


Fig. 61



PROBLEM 13. — *To draw a circular arc through three given points, as A, B, and C* (Fig. 61).

Draw a line from A to B and from B to C. Bisect AB and BC by the lines *aa* and *cc*, and prolong these lines until they intersect at *o*, which will be the centre for the arc sought. With *o* as a centre, and Ao as a radius, describe the arc ABC.

PROBLEM 14. — *To describe a circular arc passing through three given points, when the centre is not available, by means of a triangle* (Fig. 62).

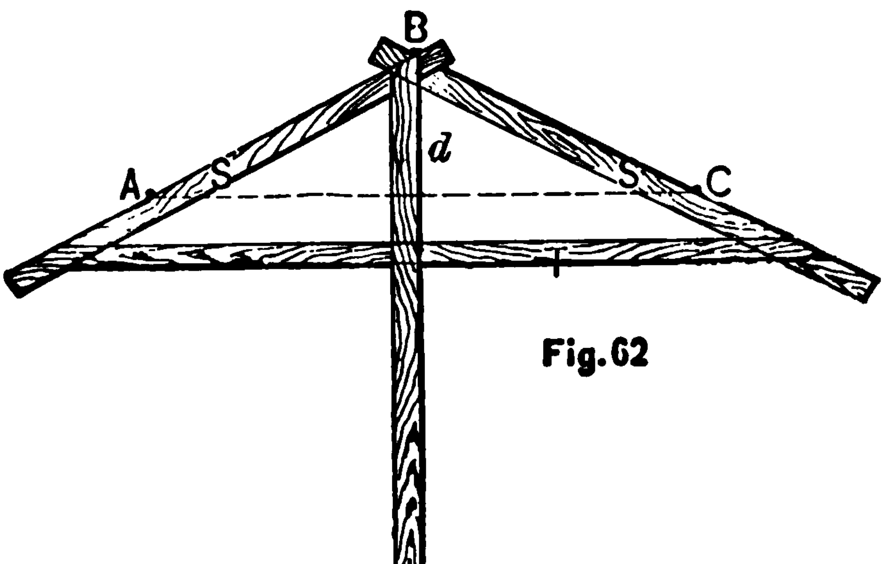


Fig. 62

Let A, B, and C be the given points. Insert two stiff pins or nails at A and C. Place two strips of wood, SS, as shown in the figure; one against A, the other against C, and inclined so that their intersection shall come at the third point, B. Fasten the strips together at their intersection, and nail a third strip, T, to their other ends, so as to make a firm triangle. Place the pencil-point at B, and, keeping the edges of the triangle against A and B, move the triangle to the left and right, and the pencil will describe the arc sought.

When the points A and C are at the same distance from B, if a strip of wood be nailed to the triangle, so that its edge de shall be at right angles to a line joining A and C as the triangle is moved one way or the other, the edge de will always point to the centre of the circle. This principle is used in the perspective linear d.

PROBLEM 15. — *To find a circular arc which shall be tangent to a given point, A, on a straight line, and pass through a given point, C, outside the line* (Fig. 63).

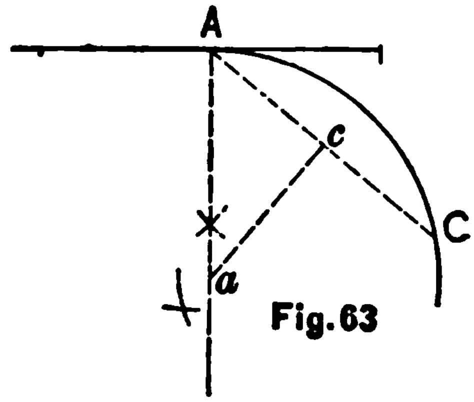


Fig. 63

Draw from A a line perpendicular to the given line. Connect A and C by a straight line, and bisect it by the perpendicular ac. The point where these two perpendiculars intersect will be the centre of the circle.

PROBLEM 16. — *To connect two parallel lines by a reversed curve composed of two circular arcs of equal radius, and tangent to the lines at given points, as A and B* (Fig. 64).

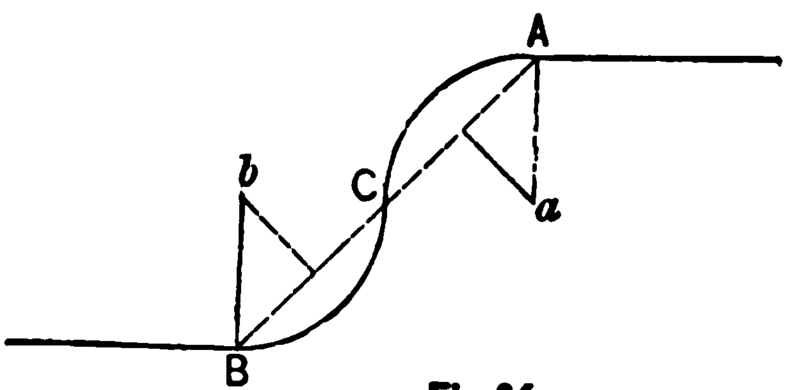


Fig. 64

Join A and B, and divide the line into two equal parts at C. Bisect CA and CB by perpendiculars. At A and B erect perpendiculars to the given lines, and the intersections a and b will be the centres of the arcs composing the required curve.

PROBLEM 17. — *On a given line, as AB, to construct a compound curve of three arcs of circles, the radii of the two side ones being equal and of a given length, and their centres in the given line; the central arc to pass through a given point, C, on the perpendicular bisecting the given line, and tangent to the other two arcs* (Fig. 65).

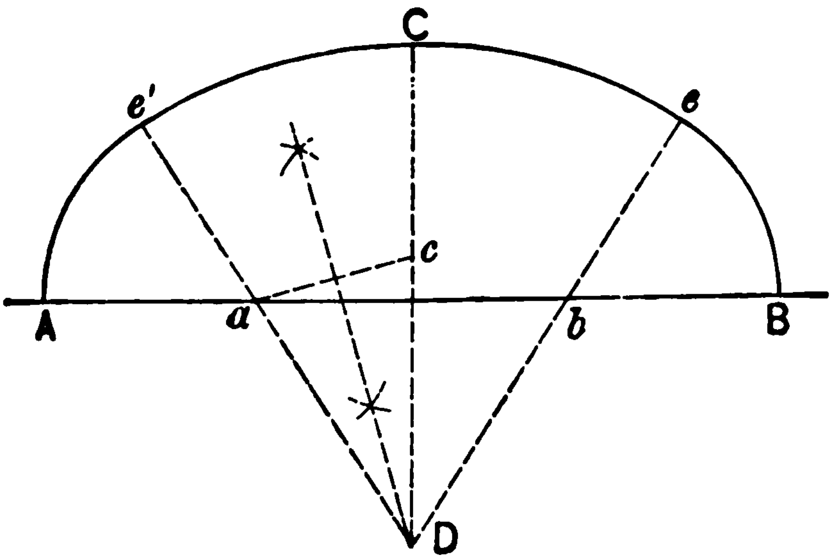


Fig. 65

Draw the perpendicular CD. Lay off Aa, Bb, and Cc, each equal to the given radius of the side arcs; join

ac; bisect *ac* by a perpendicular. The intersection of this line with the perpendicular *CD* will be the required centre of the central arc. Through *a* and *b* draw the lines *De* and *De'*; from *a* and *b*, with the given radius, equal to *Aa*, *Bb*, describe the arcs *Ae'* and *Be*; from *D* as a centre, and *CD* as a radius, describe the arc *eCe'* which completes the curve required.

PROBLEM 18. — *To construct a triangle upon a given straight line or base, the length of the two sides being given* (Fig. 66).

First (an equilateral triangle, Fig. 66a). — With the extremities *A* and *B* of the given line as centres, and *AB* as a radius, describe arcs cutting each other at *C*. Join *AC* and *BC*.

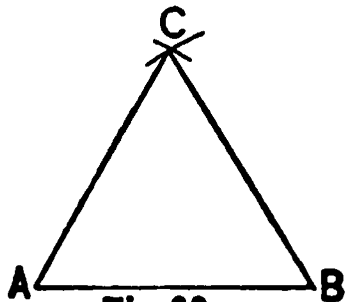


Fig. 66 a

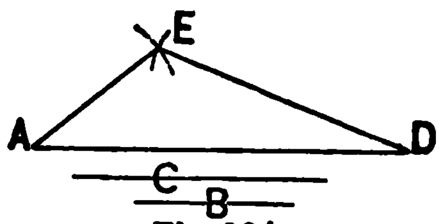


Fig. 66 b

Second (when the sides are unequal, Fig. 66b). — Let *AD* be the given base, and the other two sides be equal to *C* and *B*. With *D* as a centre, and a radius equal to *C*, describe an indefinite arc. With *A* as a centre, and *B* as a radius, describe an arc cutting the first at *E*. Join *E* with *A* and *D*, and it will give the required triangle.

PROBLEM 19. — *To describe a circle about a triangle* (Fig. 67).

Bisect two of the sides, as *AC* and *CB*, of the triangle, and at their centres erect perpendicular lines, as *ae* and *be*, intersecting at *e*. With *e* as a centre, and *eC* as a radius, describe a circle, and it will be found to pass through *A* and *B*.

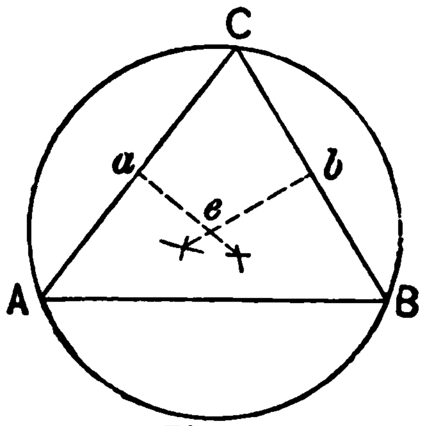


Fig. 67

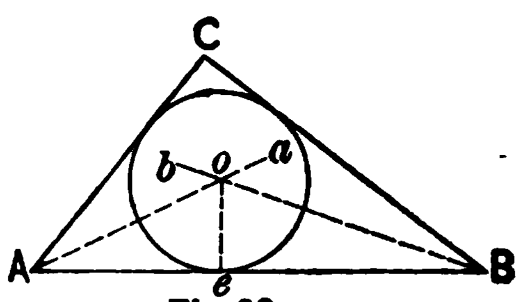


Fig. 68

PROBLEM 20. — *To inscribe a circle in a triangle* (Fig. 68).

Bisect two of the angles, *A* and *B*, of the triangle by lines cutting each other at *o*. With *o* as a centre, and *oe* as a radius, describe a circle, which will be found to just touch the other two sides.

Problem 21. — *To inscribe a square in a circle, and to describe a circle about a square* (Fig. 69).

To inscribe the square. Draw two diameters, AB and CD, at right angles to each other. Join the points A, D, B, C, and we have the inscribed square.

To describe the circle. Draw the diagonals as before, intersecting at E, and, with E as a centre and AE as a radius, describe the circle.

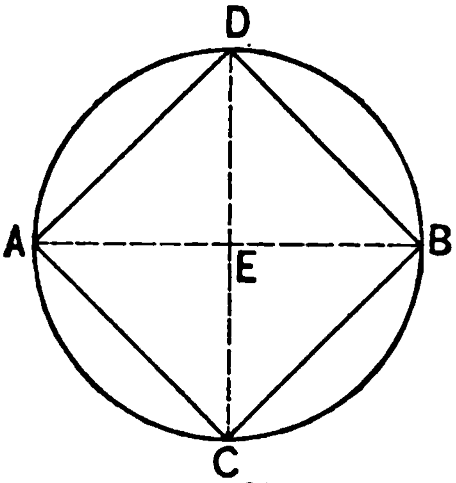


Fig. 69

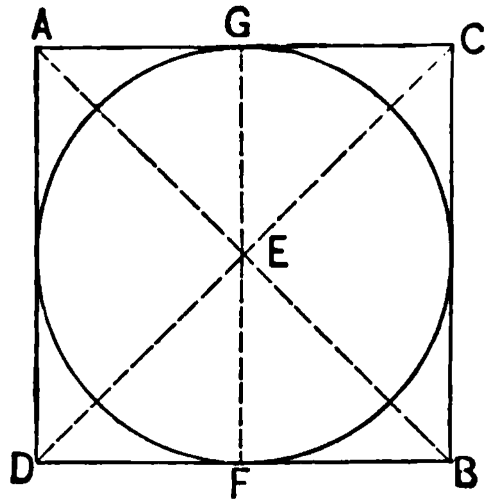


Fig. 70.

Problem 22. — *To inscribe a circle in a square, and to describe a square about a circle* (Fig. 70).

To inscribe the circle. Draw the diagonals AB and CD, intersecting at E. Draw the perpendicular EG to one of the sides. Then with E as a centre, and EG as a radius, describe a circle, which will be found to touch all four sides of the square.

To describe the square. Draw two diameters, AB and CD, at right angles to each other, and prolonged beyond the circumference. Draw the diameter GF, bisecting the angle CEA or BED. Draw lines through G and F perpendicular to GF, and terminating in the diagonals. Draw AD and CB to complete the square.

Problem 23. — *To inscribe a pentagon in a circle* (Fig. 71).

Draw two diameters, AB and CD, at right angles to each other. Bisect AO at E. With E as a centre, and EC as a radius, cut OB at F. With C as a centre, and CF as a radius, cut the circle at G and H. With these points as centres, and the same radius, cut the circle at I and J. Join I, J, H, G, and C, and we then have inscribed in the circle a regular pentagon.

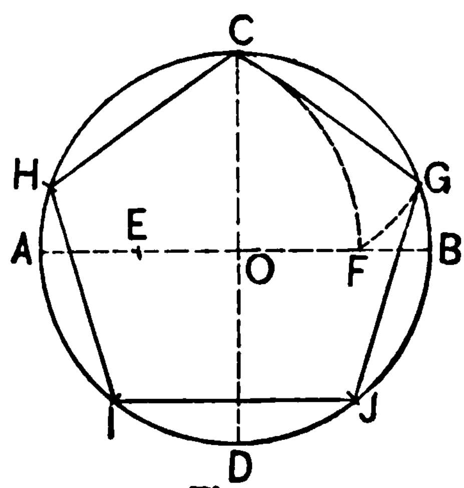


Fig. 71

PROBLEM 24.—*To inscribe a regula*

SOLUTION.—Lay off on the circ
circle six times, and connect the p

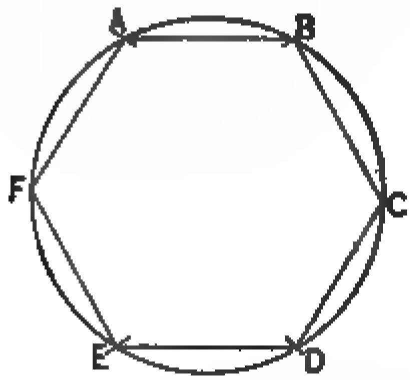


Fig. 72

F

PROBLEM 25.—*To construct a re*
straight line, AB (Fig. 73).

From A and B, with a radius equal
at O. With O as a centre, and a rad
circle, and from A and B lay off th
ference of the circle, and join the
result will be a regular hexagon.

PROBLEM 26.—*To construct a re*
straight line, AB (Fig. 74).

Produce the line AB both ways, an
and Bb, of indefinite length. Bisect
B, and make the length of the lines e
draw lines parallel to Aa, and equal in
centres G and D describe arcs, with
pendiculars Aa and Bb in F and E.

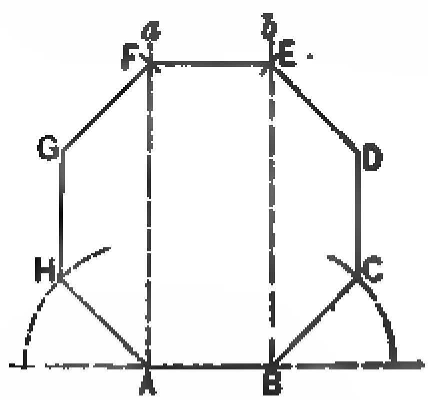


Fig. 74

PROBLEM 27.—*To make a regular o*

Draw the diagonals AD and BC,
C, and D, with a radius equal to A

sides of the square in *a*, *b*, *c*, *d*, *e*, *f*, *h*, and *i*. Join these points to complete the octagon.

PROBLEM 28. — *To inscribe a regular octagon in a circle* (Fig. 76).

Draw two diameters, *AB* and *CD*, at right angles to each other. Bisect the angles *AOD* and *AOC* by the diameters *EF* and *GH*. Join *A*, *E*, *D*, *H*, *B*, etc., for the inscribed figure.

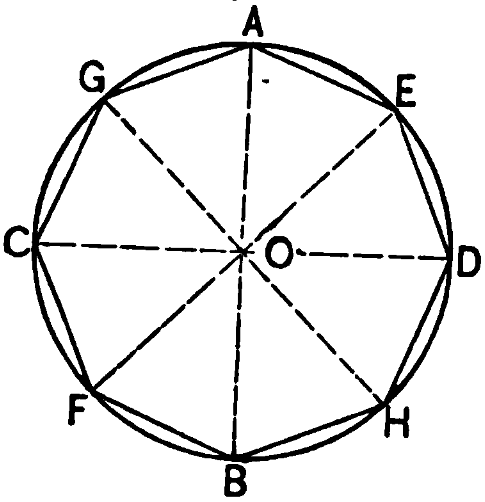


Fig.76

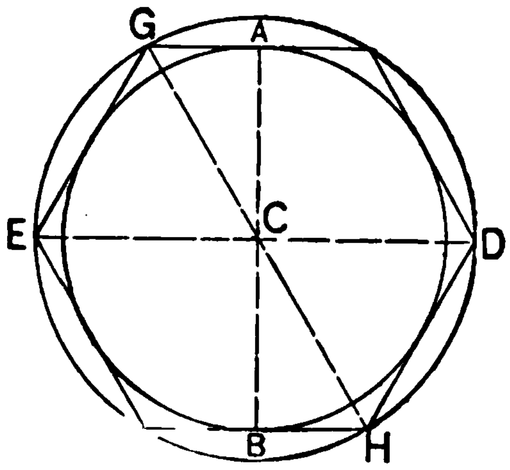


Fig.77

PROBLEM 29. — *To inscribe a circle within a regular polygon.*

First (when the polygon has an even number of sides, as in Fig. 77). — Bisect two opposite sides at *A* and *B*, and draw *AB*, and bisect it at *C* by a diagonal, *DE*, drawn between two opposite angles. With the radius *CA* describe the circle as required.

Second (when the number of sides is odd, as in Fig. 78). — Bisect two of the sides at *A* and *B*, and draw lines, *AE* and *BD*, to the opposite angles, intersecting at *C*. With *C* as a centre, and *CA* as a radius, describe the circle as required.

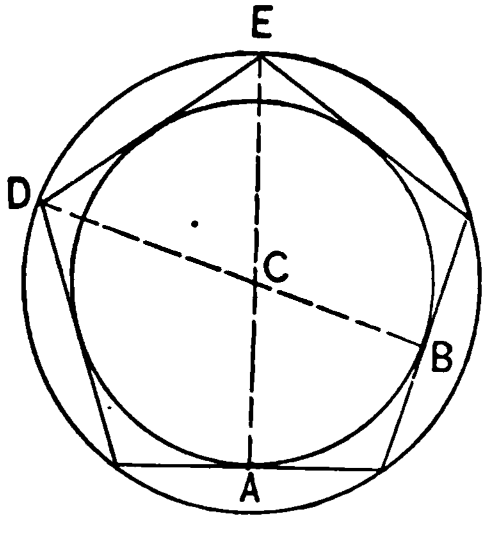


Fig.78

PROBLEM 30. — *To describe a circle without a regular polygon.*

When the number of the sides is even, draw two diagonals from opposite angles, as *ED* and *GH* (Fig. 77), intersecting at *C*; and from *C*, with *CD* as a radius, describe the circle required.

When the number of sides is odd, find the centre, *C*, as in last problem; and with *C* as a centre, and *CD* (Fig. 78) as a radius, describe the circle required.

PROBLEMS ON THE ELLIPSE, THE PARA THE HYPERBOLA, AND THE CYCLOI

The Ellipse.

PROBLEM 31.—*To describe an ellipse, the length and bre the two axes, being given.*

1ST M (Fig. 79, t axes, AB a being gi On AB a as diamete from the centre, O, d the circles and $CLDK$ any co number of on the cir ence of th circle, as b'', etc., an them dra to the cen cutting th circle at th

A

G

Fig.79

a, a', a'', etc., respectively. From the points b, b', etc., dra parallel to the shorter axis; and from the points a, a', etc

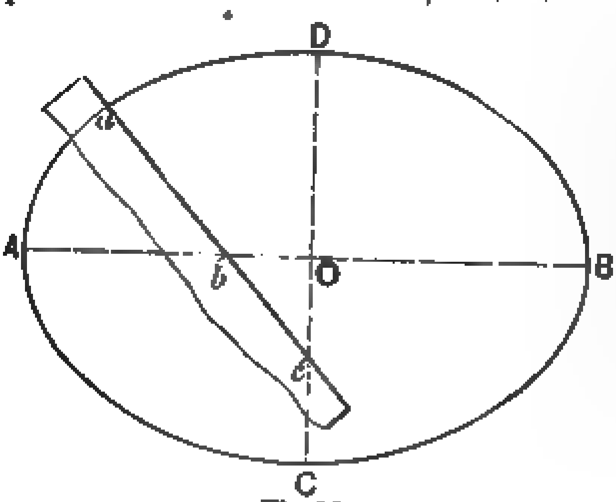


Fig.80

lines parallel to the axis, and intersect first set of lines c'', etc. These last will be points in lipse, and, by obta sufficient number o the ellipse can ea drawn.

2D METHOD (F —Take the straig of a stiff piece of cardboard, or wo from some point, as a, mark off ab equal to half the shorte

eter, and ac equal to half the longer diameter. Place the straight edge so that the point b shall be on the longer diameter, and the point c on the shorter: then will the point a be over a point in the ellipse. Make on the paper a dot at a, and move the slip around, always keeping the points b and c over the major and minor axes. In this way any number of points in the ellipse may be obtained, which may be connected by a curve drawn freehand.

3D METHOD (Fig. 81, given the two axes AB and CD.)—From the point D as a centre, and a radius AO, equal to one-half of AB, describe an arc cutting AB at F and F'. These two points are called the foci of the ellipse. [One property of the ellipse is, that the sum of the distances of any two points on the circumference from the foci is the same. Thus $F'D + DF = F'E + EF$ or $F'G + GF$.] Fix a couple of pins into the axis AB at F and F', and loop a thread or cord upon them equal in length, when fastened to the pins, to AB, so as, when stretched as per dotted line FDF', just to reach the extremity D of the short axis. Place a pencil-point inside the chord, as at E, and move the pencil along, always keeping the cord stretched tight. In this way the pencil will trace the outline of the ellipse.

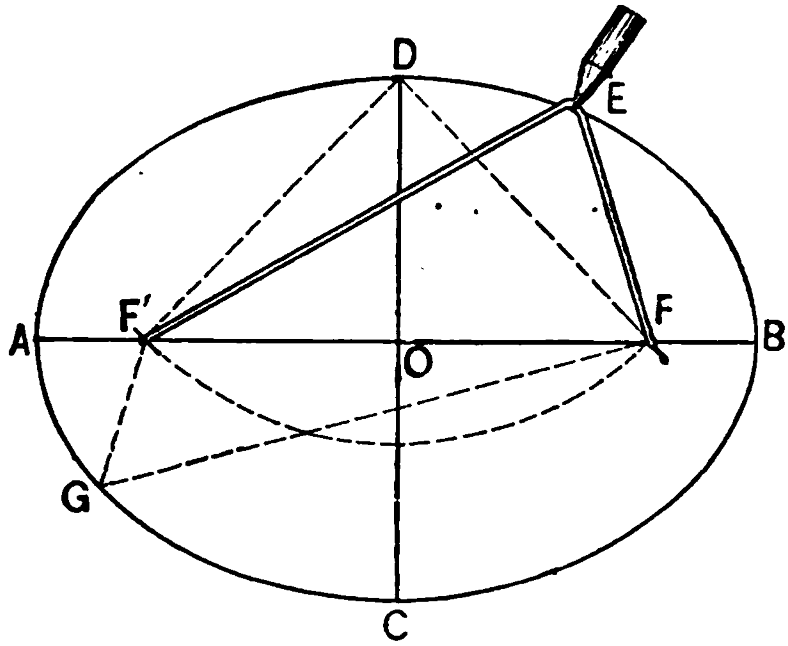


Fig. 81

PROBLEM 32.—*To draw a tangent to an ellipse at a given point on the curve* (Fig. 82).

Let it be required to draw a tangent at the point E on the ellipse shown in Fig. 82. First find the foci F and F', as in the third method for describing an ellipse, then from

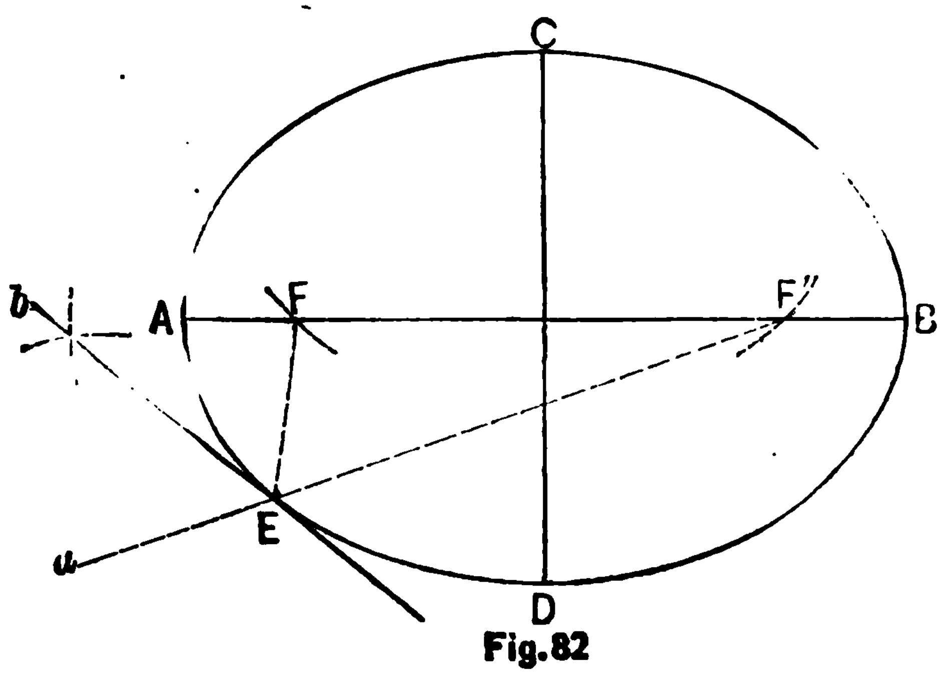


Fig. 82

E draw lines *EF* and *EF′*. Prolong *EF′* to *a*, so that *Ea* shall equal *EF*. Bisect the angle *aEF* as at *b*, and through *b* draw a line touching the curve at *E*. This line will be the tangent required. If it were desired to draw a line normal to the curve at *E*, as, for instance, the joint of an elliptical arch, bisect the angle *FEF′*, and draw the bisecting line through *E*, and it will be the normal to the curve, and the proper line for the joint of an elliptical arch at that point.

PROBLEM 33. — *To draw a tangent to an ellipse from a given point without the curve* (Fig. 83).

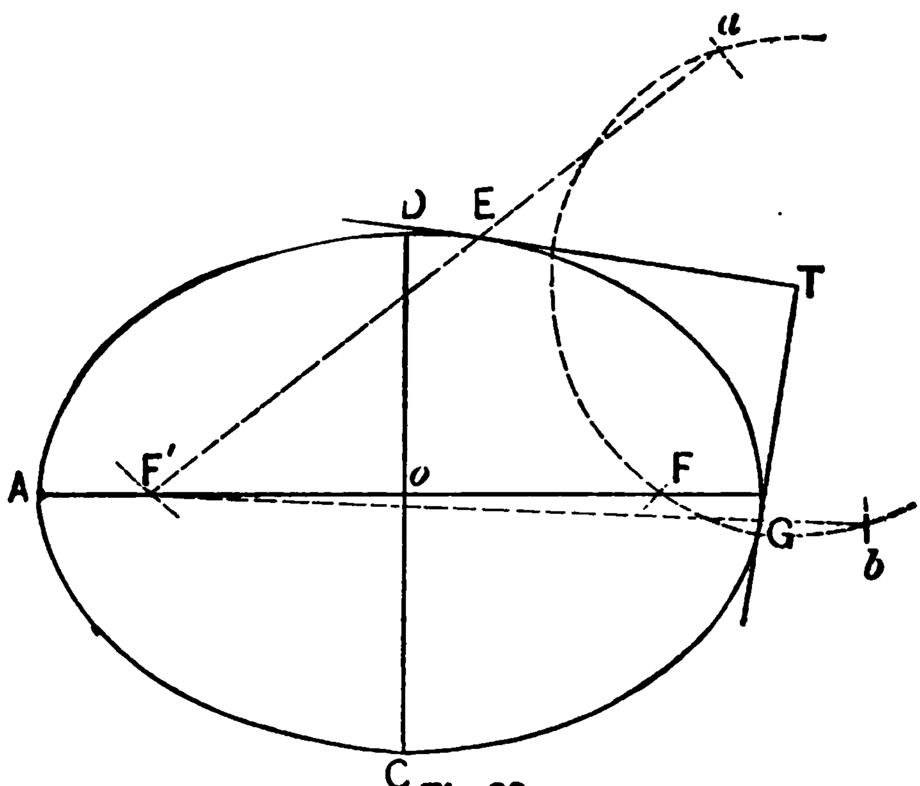


Fig. 83

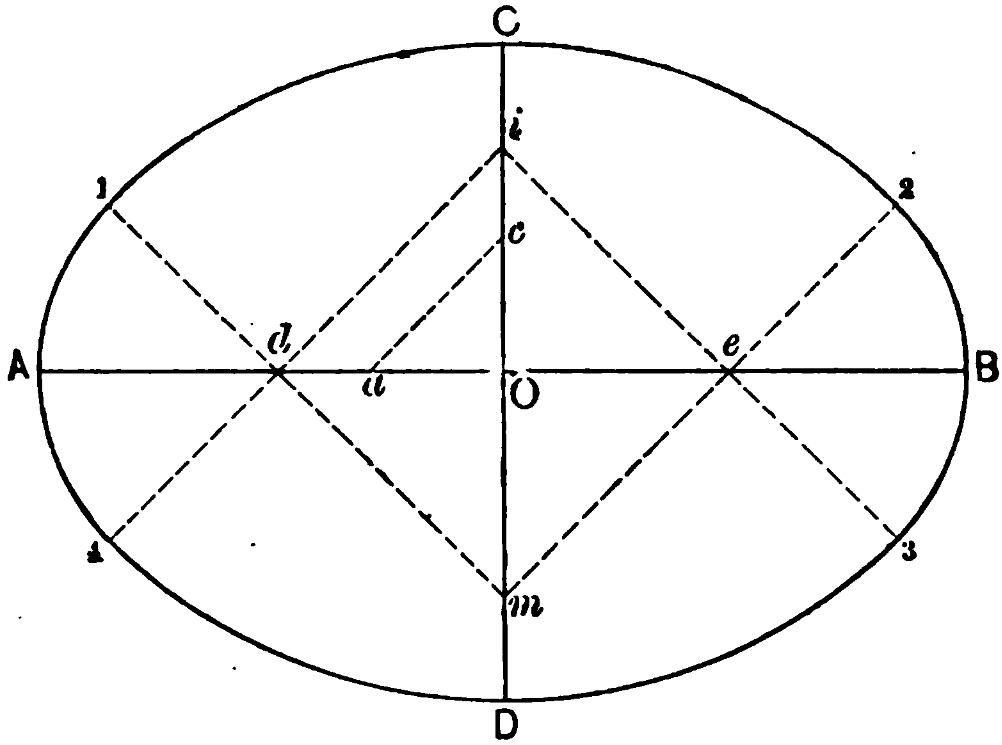
From the point *T* as a centre, and a radius equal to the distance to the nearer focus *F*, describe a circle. From *F′* as a centre, and a radius equal to the length of the longer axis, describe arcs cutting the circle just described at *a* and *b*. Draw lines from *F′* to *a* and *b*, cutting the circumference of the ellipse at *E* and *G*. Draw lines from *T* through *E* and *G*, and they will be the tangents required.

PROBLEM 34. — *To describe an ellipse approximately, by means of circular arcs.*

First (with arcs of two radii, Fig. 84). — Take half the difference of the two axes *AB* and *CD*, and set it off from the centre *O* to *a* and *c* on *OA* and *OC*; draw *ac*, and set off half *ac* to *d*; draw *di* parallel to *ac*; set off *Oe* equal to *Od*; join *ei*, and draw *em* and *dm* parallels to *di* and *ie*. On *m* as a centre, with a radius *mC*, describe an arc through *C*, terminating in 1 and 2; and with *i* as a centre, and *id* as a radius, describe an arc through *D*, terminating in points 3 and 4. On *d* and *e* as centres describe arcs through *A* and *B*, connecting the points 1 and 4, 2 and 3. The four arcs thus de-

scribed form approximately an ellipse. This method does not apply satisfactorily when the conjugate axis is less than two-thirds of the transverse axis.

Fig. 84



Second (with arcs of three radii, Fig. 85). — On the transverse axis AB draw the rectangle $AGEB$, equal in height to OC, half

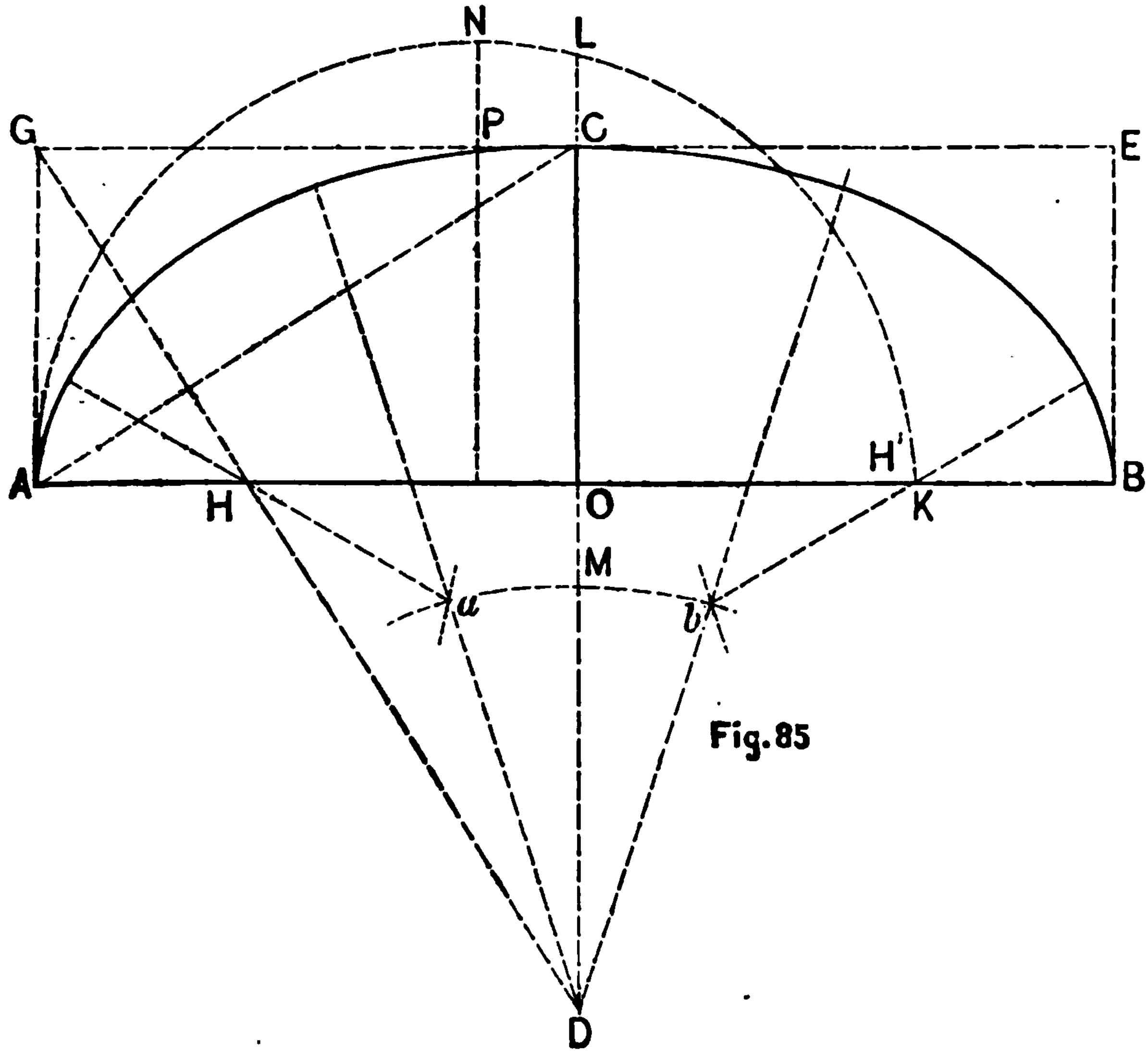


Fig. 85

the conjugate axis. Draw GD perpendicular to AC. Set off OK equal to OC, and on AK as a diameter describe the semicircle

ANK. Draw a radius parallel to *OC*, intersecting the semicircle at *N*, and the line *GE* at *P*. Extend *OC* to *L* and to *D*. Set off *OM* equal to *PN*, and on *D* as a centre, with a radius *DM*, describe an arc. From *A* and *B* as centres, with a radius *OL*, intersect this arc at *a* and *b*. The points *H*, *a*, *D*, *b*, *H′*, are the centres of the arcs required. Produce the lines *aH*, *Da*, *Db*, *bH′*, and the spaces enclosed determine the lengths of each arc. This process works well for nearly all ellipses. It is employed in striking out vaults, stone arches, and bridges.

NOTE.—In this example the point *H′* happens to coincide with the point *K*, but this need not necessarily be the case.

The Parabola.

PROBLEM 35.—*To construct a parabola when the vertex A, the axis AB, and a point, M, of the curve, are given* (Fig. 86).

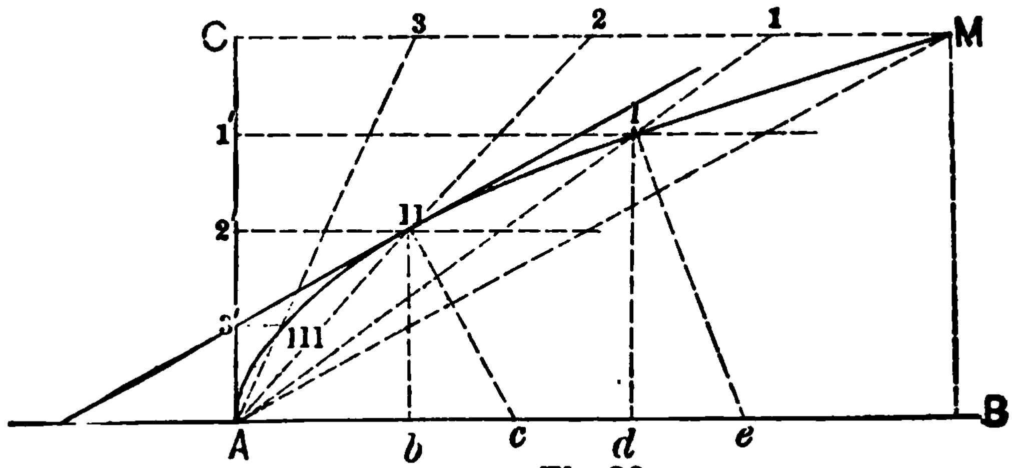


Fig. 86

Construct the rectangle *ABMC*. Divide *MC* into any number of equal parts, four for instance. Divide *AC* in like manner. Connect *A*1, *A*2, and *A*3. Through 1′, 2′, 3′, draw parallels to the axis. The intersections I, II, and III, of these lines, are points in the required curve.

PROBLEM 36.—*To draw a tangent to a given point,* II, *of the parabola* (Fig. 86).

From the given point II let fall a perpendicular on the axis at *b*. Extend the axis to the left of *A*. Make *Aa* equal to *Ab*. Draw *a*II, and it is the tangent required.

The lines perpendicular to the tangent are called normals. *To find the normal to any point* I, *having the tangent to any other point,* II. Draw the normal II*c*. From I let fall a perpendicular I*d*, on the axis *AB*. Lay off *de* equal to *bc*. Connect I*e*, and we have the normal required. The tangent may be drawn at I by laying off a perpendicular to the normal I*e* at I.

The Hyperbola.

The hyperbola possesses the characteristic that if, from any point, P, two straight lines be drawn to two fixed points, F and F', the foci, their *difference* shall always be the same.

PROBLEM 37. — *To describe an hyperbola through a given vertex, a, with the given difference ab, and one of the foci, F* (Fig 87).

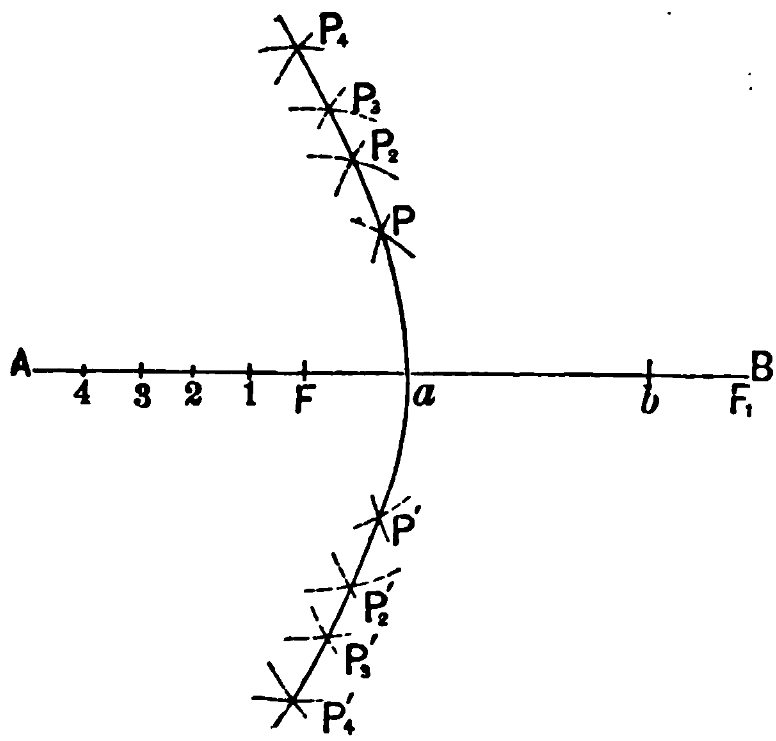


Fig. 87.

Draw the axis of the hyperbola AB, with the given distance ab and the focus F marked on it. From b lay off bF_1 equal to aF for the other focus. Take any point, as 1 on AB, and with $a1$ as a radius, and F as a centre, describe two short arcs above and below the axis. With $b1$ as a radius, and F' as a centre, describe arcs cutting those just described at P and P'. Take several points, as 2, 3, and 4, and obtain the corresponding points P_2, P_3, and P_4 in the same way. Join these points with a curved line, and it will be an hyperbola.

To draw a tangent to any point of an hyperbola, draw lines from the given point to each of the foci, and bisect the angle thus formed. The bisecting line will be the tangent required.

The Cycloid.

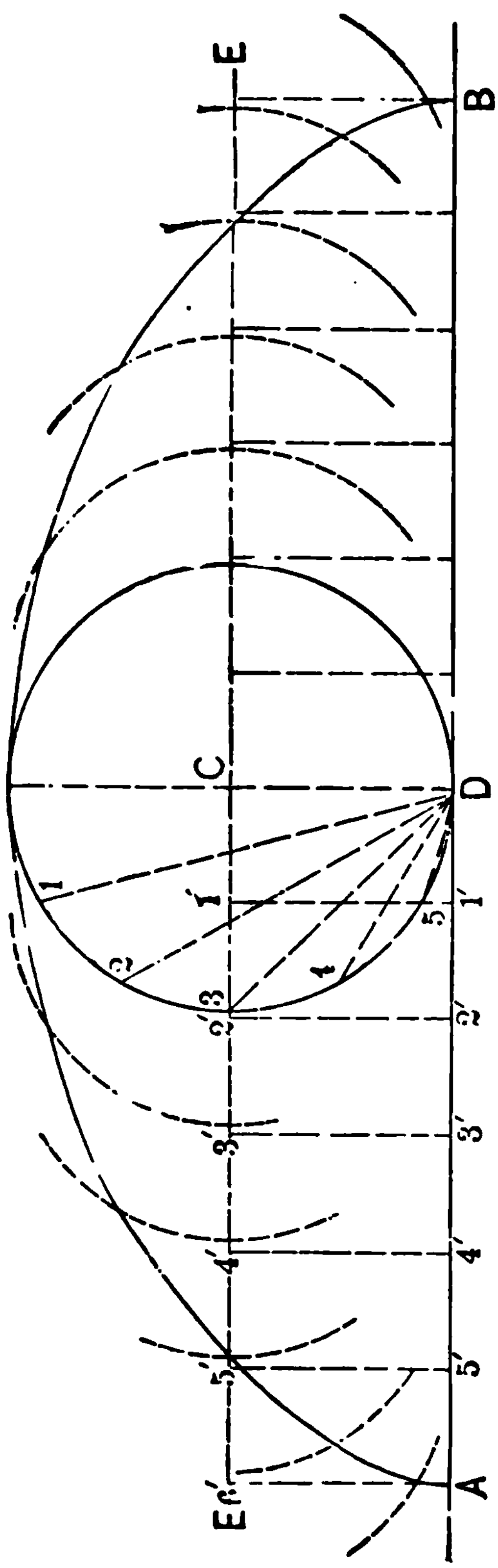


Fig. 88.

The *cycloid* is the curve described by a point in the circumference of a circle rolling in a straight line.

PROBLEM 38. — *To describe a cycloid* (Fig. 88).

Draw the straight line *AB* as the base. Describe the generating circle tangent to this line at the centre, and through the centre of the circle, *C*, draw the line *EE* parallel to the base. Let fall a perpendicular from *C* upon the base. Divide the semi-circumference into any number of equal parts, for instance, six. Lay off on *AB* and *CE* distances *C*1′, 1′2′, etc., equal to the divisions of the circumference. Draw the chords *D*1, *D*2, etc. From the points 1′, 2′, 3′, on the line *CE*, with radii equal to the generating circle, describe arcs. From the points 1′, 2′, 3′, 4′, 5′, on the line *BA*, and with radii equal respectively to the chords *D*1, *D*2, *D*3, *D*4, *D*5, describe arcs cutting the preceding, and the intersections will be points of the curve required.

TABLE OF CHORDS; Radius = 1.0000.

Table of Chords; Radius = 1.0000 (*continued*).

M.	11°	12°	13°	14°	15°	16°	17°	18°	19°	20°	21°	M.
0′	.1917	.2091	.2264	.2437	.2611	.2783	.2956	.3129	.3301	.3473	.3645	0′
1	.1920	.2093	.2267	.2440	.2613	.2786	.2959	.3132	.3304	.3476	.3648	1
2	.1923	.2096	.2270	.2443	.2616	.2789	.2962	.3134	.3307	.3479	.3650	2
3	.1926	.2099	.2273	.2446	.2619	.2792	.2965	.3137	.3310	.3482	.3653	3
4	.1928	.2102	.2276	.2449	.2622	.2795	.2968	.3140	.3312	.3484	.3656	4
5	.1931	.2105	.2279	.2452	.2625	.2798	.2971	.3143	.3315	.3487	.3659	5
6	.1934	.2108	.2281	.2455	.2628	.2801	.2973	.3146	.3318	.3490	.3662	6
7	.1937	.2111	.2284	.2458	.2631	.2804	.2976	.3149	.3321	.3493	.3665	7
8	.1940	.2114	.2287	.2460	.2634	.2807	.2979	.3152	.3324	.3496	.3668	8
9	.1943	.2117	.2290	.2463	.2636	.2809	.2982	.3155	.3327	.3499	.3670	9
10	.1946	.2119	.2293	.2466	.2639	.2812	.2985	.3157	.3330	.3502	.3673	10
11	.1949	.2122	.2296	.2469	.2642	.2815	.2988	.3160	.3333	.3504	.3676	11
12	.1952	.2125	.2299	.2472	.2645	.2818	.2991	.3163	.3335	.3507	.3679	12
13	.1955	.2128	.2302	.2475	.2648	.2821	.2994	.3166	.3338	.3510	.3682	13
14	.1957	.2131	.2305	.2478	.2651	.2824	.2996	.3169	.3341	.3513	.3685	14
15	.1960	.2134	.2307	.2481	.2654	.2827	.2999	.3172	.3344	.3516	.3688	15
16	.1963	.2137	.2310	.2484	.2657	.2830	.3002	.3175	.3347	.3519	.3690	16
17	.1966	.2140	.2313	.2486	.2660	.2832	.3005	.3178	.3350	.3522	.3693	17
18	.1969	.2143	.2316	.2489	.2662	.2835	.3008	.3180	.3353	.3525	.3696	18
19	.1972	.2146	.2319	.2492	.2665	.2838	.3011	.3183	.3355	.3527	.3699	19
20	.1975	.2148	.2322	.2495	.2668	.2841	.3014	.3186	.3358	.3530	.3702	20
21	.1978	.2151	.2325	.2498	.2671	.2844	.3017	.3189	.3361	.3533	.3705	21
22	.1981	.2154	.2328	.2501	.2674	.2847	.3019	.3192	.3364	.3536	.3708	22
23	.1983	.2157	.2331	.2504	.2677	.2850	.3022	.3195	.3367	.3539	.3710	23
24	.1986	.2160	.2333	.2507	.2680	.2853	.3025	.3198	.3370	.3542	.3713	24
25	.1989	.2163	.2336	.2510	.2683	.2855	.3028	.3200	.3373	.3545	.3716	25
26	.1992	.2166	.2339	.2512	.2685	.2858	.3031	.3203	.3376	.3547	.3719	26
27	.1995	.2169	.2342	.2515	.2688	.2861	.3034	.3206	.3378	.3550	.3722	27
28	.1998	.2172	.2345	.2518	.2691	.2864	.3037	.3209	.3381	.3553	.3725	28
29	.2001	.2174	.2348	.2521	.2694	.2867	.3040	.3212	.3384	.3556	.3728	29
30	.2004	.2177	.2351	.2524	.2697	.2870	.3042	.3215	.3387	.3559	.3730	30
31	.2007	.2180	.2354	.2527	.2700	.2873	.3045	.3218	.3390	.3562	.3733	31
32	.2010	.2183	.2357	.2530	.2703	.2876	.3048	.3221	.3393	.3565	.3736	32
33	.2012	.2186	.2359	.2533	.2706	.2878	.3051	.3223	.3396	.3567	.3739	33
34	.2015	.2189	.2362	.2536	.2709	.2881	.3054	.3226	.3398	.3570	.3742	34
35	.2018	.2192	.2365	.2538	.2711	.2884	.3057	.3229	.3401	.3573	.3745	35
36	.2021	.2195	.2368	.2541	.2714	.2887	.3060	.3232	.3404	.3576	.3748	36
37	.2024	.2198	.2371	.2544	.2717	.2890	.3063	.3235	.3407	.3579	.3750	37
38	.2027	.2200	.2374	.2547	.2720	.2893	.3065	.3238	.3410	.3582	.3753	38
39	.2030	.2203	.2377	.2550	.2723	.2896	.3068	.3241	.3413	.3585	.3756	39
40	.2033	.2206	.2380	.2553	.2726	.2899	.3071	.3244	.3416	.3587	.3759	40
41	.2036	.2209	.2383	.2556	.2729	.2902	.3074	.3246	.3419	.3590	.3762	41
42	.2038	.2212	.2385	.2559	.2732	.2904	.3077	.3249	.3421	.3593	.3765	42
43	.2041	.2215	.2388	.2561	.2734	.2907	.3080	.3252	.3424	.3596	.3768	43
44	.2044	.2218	.2391	.2564	.2737	.2910	.3083	.3255	.3427	.3599	.3770	44
45	.2047	.2221	.2394	.2567	.2740	.2913	.3086	.3258	.3430	.3602	.3773	45
46	.2050	.2224	.2397	.2570	.2743	.2916	.3088	.3261	.3433	.3605	.3776	46
47	.2053	.2226	.2400	.2573	.2746	.2919	.3091	.3264	.3436	.3608	.3779	47
48	.2056	.2229	.2403	.2576	.2749	.2922	.3094	.3267	.3439	.3610	.3782	48
49	.2059	.2232	.2406	.2579	.2752	.2925	.3097	.3269	.3441	.3613	.3785	49
50	.2062	.2235	.2409	.2582	.2755	.2927	.3100	.3272	.3444	.3616	.3788	50
51	.2065	.2238	.2411	.2585	.2758	.2930	.3103	.3275	.3447	.3619	.3790	51
52	.2067	.2241	.2414	.2587	.2760	.2933	.3106	.3278	.3450	.3622	.3793	52
53	.2070	.2244	.2417	.2590	.2763	.2936	.3109	.3281	.3453	.3625	.3796	53
54	.2073	.2247	.2420	.2593	.2766	.2939	.3111	.3284	.3456	.3628	.3799	54
55	.2076	.2250	.2423	.2596	.2769	.2942	.3114	.3287	.3459	.3630	.3802	55
56	.2079	.2253	.2426	.2599	.2772	.2945	.3117	.3289	.3462	.3633	.3805	56
57	.2082	.2256	.2429	.2602	.2775	.2948	.3120	.3292	.3464	.3636	.3808	57
58	.2085	.2258	.2432	.2605	.2778	.2950	.3123	.3295	.3467	.3639	.3810	58
59	.2088	.2261	.2434	.2608	.2781	.2953	.3126	.3298	.3470	.3642	.3813	59
60	.2091	.2264	.2437	.2611	.2783	.2956	.3129	.3301	.3473	.3645	.3816	60

Table of Chords; Radius = 1.0000 (*continued*).

M.	22°	23°	24°	25°	26°	27°	28°	29°	30°	31°	32°	M.
0′	.3816	.3987	.4158	.4329	.4499	.4669	.4838	.5008	.5176	.5345	.5513	0′
1	.3819	.3990	.4161	.4332	.4502	.4672	.4841	.5010	.5179	.5348	.5516	1
2	.3822	.3993	.4164	.4334	.4505	.4675	.4844	.5013	.5182	.5350	.5518	2
3	.3825	.3996	.4167	.4337	.4508	.4677	.4847	.5016	.5185	.5353	.5521	3
4	.3828	.3999	.4170	.4340	.4510	.4680	.4850	.5019	.5188	.5356	.5524	4
5	.3830	.4002	.4172	.4343	.4513	.4683	.4853	.5022	.5190	.5359	.5527	5
6	.3833	.4004	.4175	.4346	.4516	.4686	.4855	.5024	.5193	.5362	.5530	6
7	.3836	.4007	.4178	.4349	.4519	.4689	.4858	.5027	.5196	.5364	.5532	7
8	.3839	.4010	.4181	.4352	.4522	.4692	.4861	.5030	.5199	.5367	.5535	8
9	.3842	.4013	.4184	.4354	.4525	.4694	.4864	.5033	.5202	.5370	.5538	9
10	.3845	.4016	.4187	.4357	.4527	.4697	.4867	.5036	.5204	.5373	.5541	10
11	.3848	.4019	.4190	.4360	.4530	.4700	.4869	.5039	.5207	.5376	.5543	11
12	.3850	.4022	.4192	.4363	.4533	.4703	.4872	.5041	.5210	.5378	.5546	12
13	.3853	.4024	.4195	.4366	.4536	.4706	.4875	.5044	.5213	.5381	.5549	13
14	.3856	.4027	.4198	.4369	.4539	.4708	.4878	.5047	.5216	.5384	.5552	14
15	.3859	.4030	.4201	.4371	.4542	.4711	.4881	.5050	.5219	.5387	.5555	15
16	.3862	.4033	.4204	.4374	.4544	.4714	.4884	.5053	.5221	.5390	.5557	16
17	.3865	.4036	.4207	.4377	.4547	.4717	.4886	.5055	.5224	.5392	.5560	17
18	.3868	.4039	.4209	.4380	.4550	.4720	.4889	.5058	.5227	.5395	.5563	18
19	.3870	.4042	.4212	.4383	.4553	.4723	.4892	.5061	.5230	.5398	.5566	19
20	.3873	.4044	.4215	.4386	.4556	.4725	.4895	.5064	.5233	.5401	.5569	20
21	.3876	.4047	.4218	.4388	.4559	.4728	.4898	.5067	.5235	.5404	.5571	21
22	.3879	.4050	.4221	.4391	.4561	.4731	.4901	.5070	.5238	.5406	.5574	22
23	.3882	.4053	.4224	.4394	.4564	.4734	.4903	.5072	.5241	.5409	.5577	23
24	.3885	.4056	.4226	.4397	.4567	.4737	.4906	.5075	.5244	.5412	.5580	24
25	.3888	.4059	.4229	.4400	.4570	.4740	.4909	.5078	.5247	.5415	.5583	25
26	.3890	.4061	.4232	.4403	.4573	.4742	.4912	.5081	.5249	.5418	.5585	26
27	.3893	.4064	.4235	.4405	.4576	.4745	.4915	.5084	.5252	.5420	.5588	27
28	.3896	.4067	.4238	.4408	.4578	.4748	.4917	.5086	.5255	.5423	.5591	28
29	.3899	.4070	.4241	.4411	.4581	.4751	.4920	.5089	.5258	.5426	.5594	29
30	.3902	.4073	.4244	.4414	.4584	.4754	.4923	.5092	.5261	.5429	.5597	30
31	.3905	.4076	.4246	.4417	.4587	.4757	.4926	.5095	.5263	.5432	.5599	31
32	.3908	.4079	.4249	.4420	.4590	.4759	.4929	.5098	.5266	.5434	.5602	32
33	.3910	.4081	.4252	.4422	.4593	.4762	.4932	.5100	.5269	.5437	.5605	33
34	.3913	.4084	.4255	.4425	.4595	.4765	.4934	.5103	.5272	.5440	.5608	34
35	.3916	.4087	.4258	.4428	.4598	.4768	.4937	.5106	.5275	.5443	.5611	35
36	.3919	.4090	.4261	.4431	.4601	.4771	.4940	.5109	.5277	.5446	.5613	36
37	.3922	.4093	.4263	.4434	.4604	.4773	.4943	.5112	.5280	.5448	.5616	37
38	.3925	.4096	.4266	.4437	.4607	.4776	.4946	.5115	.5283	.5451	.5619	38
39	.3927	.4098	.4269	.4439	.4609	.4779	.4948	.5117	.5286	.5454	.5622	39
40	.3930	.4101	.4272	.4442	.4612	.4782	.4951	.5120	.5289	.5457	.5625	40
41	.3933	.4104	.4275	.4445	.4615	.4785	.4954	.5123	.5291	.5460	.5627	41
42	.3936	.4107	.4278	.4448	.4618	.4788	.4957	.5126	.5294	.5462	.5630	42
43	.3939	.4110	.4280	.4451	.4621	.4790	.4960	.5129	.5297	.5465	.5633	43
44	.3942	.4113	.4283	.4454	.4624	.4793	.4963	.5131	.5300	.5468	.5636	44
45	.3945	.4116	.4286	.4456	.4626	.4796	.4965	.5134	.5303	.5471	.5638	45
46	.3947	.4118	.4289	.4459	.4629	.4799	.4968	.5137	.5306	.5474	.5641	46
47	.3950	.4121	.4292	.4462	.4632	.4802	.4971	.5140	.5308	.5476	.5644	47
48	.3953	.4124	.4295	.4465	.4635	.4805	.4974	.5143	.5311	.5479	.5647	48
49	.3956	.4127	.4298	.4468	.4638	.4807	.4977	.5145	.5314	.5482	.5650	49
50	.3959	.4130	.4300	.4471	.4641	.4810	.4979	.5148	.5317	.5485	.5652	50
51	.3962	.4133	.4303	.4474	.4643	.4813	.4982	.5151	.5320	.5488	.5655	51
52	.3965	.4135	.4306	.4476	.4646	.4816	.4985	.5154	.5322	.5490	.5658	52
53	.3967	.4138	.4309	.4479	.4649	.4819	.4988	.5157	.5325	.5493	.5661	53
54	.3970	.4141	.4312	.4482	.4652	.4822	.4991	.5160	.5328	.5496	.5664	54
55	.3973	.4144	.4315	.4485	.4655	.4824	.4994	.5162	.5331	.5499	.5666	55
56	.3976	.4147	.4317	.4488	.4658	.4827	.4996	.5165	.5334	.5502	.5669	56
57	.3979	.4150	.4320	.4491	.4660	.4830	.4999	.5168	.5336	.5504	.5672	57
58	.3982	.4153	.4323	.4493	.4663	.4833	.5002	.5171	.5339	.5507	.5675	58
59	.3985	.4155	.4326	.4496	.4666	.4836	.5005	.5174	.5342	.5510	.5678	59
60	.3987	.4158	.4329	.4499	.4669	.4838	.5008	.5176	.5345	.5513	.5680	60

Table of Chords; Radius = 1.0000 (*continued*).

M.	33°	34°	35°	36°	37°	38°	39°	40°	41°	42°	43°	M.
0′	.5680	.5847	.6014	.6180	.6346	.6511	.6676	.6840	.7004	.7167	.7330	0′
1	.5683	.5850	.6017	.6183	.6349	.6514	.6679	.6843	.7007	.7170	.7333	1
2	.5686	.5853	.6020	.6186	.6352	.6517	.6682	.6846	.7010	.7173	.7335	2
3	.5689	.5856	.6022	.6189	.6354	.6520	.6684	.6849	.7012	.7176	.7338	3
4	.5691	.5859	.6025	.6191	.6357	.6522	.6687	.6851	.7015	.7178	.7341	4
5	.5694	.5861	.6028	.6194	.6360	.6525	.6690	.6854	.7018	.7181	.7344	5
6	.5697	.5864	.6031	.6197	.6363	.6528	.6693	.6857	.7020	.7184	.7346	6
7	.5700	.5867	.6034	.6200	.6365	.6531	.6695	.6860	.7023	.7186	.7349	7
8	.5703	.5870	.6036	.6202	.6368	.6533	.6698	.6862	.7026	.7189	.7352	8
9	.5705	.5872	.6039	.6205	.6371	.6536	.6701	.6865	.7029	.7192	.7354	9
10	.5708	.5875	.6042	.6208	.6374	.6539	.6704	.6868	.7031	.7195	.7357	10
11	.5711	.5878	.6045	.6211	.6376	.6542	.6706	.6870	.7034	.7197	.7360	11
12	.5714	.5881	.6047	.6214	.6379	.6544	.6709	.6873	.7037	.7200	.7362	12
13	.5717	.5884	.6050	.6216	.6382	.6547	.6712	.6876	.7040	.7203	.7365	13
14	.5719	.5886	.6053	.6219	.6385	.6550	.6715	.6879	.7042	.7205	.7368	14
15	.5722	.5889	.6056	.6222	.6387	.6553	.6717	.6881	.7045	.7208	.7371	15
16	.5725	.5892	.6058	.6225	.6390	.6555	.6720	.6884	.7048	.7211	.7373	16
17	.5728	.5895	.6061	.6227	.6393	.6558	.6723	.6887	.7050	.7214	.7376	17
18	.5730	.5897	.6064	.6230	.6396	.6561	.6725	.6890	.7053	.7216	.7379	18
19	.5733	.5900	.6067	.6233	.6398	.6564	.6728	.6892	.7056	.7219	.7381	19
20	.5736	.5903	.6070	.6236	.6401	.6566	.6731	.6895	.7059	.7222	.7384	20
21	.5739	.5906	.6072	.6238	.6404	.6569	.6734	.6898	.7061	.7224	.7387	21
22	.5742	.5909	.6075	.6241	.6407	.6572	.6736	.6901	.7064	.7227	.7390	22
23	.5744	.5911	.6078	.6244	.6410	.6575	.6739	.6903	.7067	.7230	.7392	23
24	.5747	.5914	.6081	.6247	.6412	.6577	.6742	.6906	.7069	.7232	.7395	24
25	.5750	.5917	.6083	.6249	.6415	.6580	.6745	.6909	.7072	.7235	.7398	25
26	.5753	.5920	.6086	.6252	.6418	.6583	.6747	.6911	.7075	.7238	.7400	26
27	.5756	.5922	.6089	.6255	.6421	.6586	.6750	.6914	.7078	.7241	.7403	27
28	.5758	.5925	.6092	.6258	.6423	.6588	.6753	.6917	.7080	.7243	.7406	28
29	.5761	.5928	.6095	.6260	.6426	.6591	.6756	.6920	.7083	.7246	.7408	29
30	.5764	.5931	.6097	.6263	.6429	.6594	.6758	.6922	.7086	.7249	.7411	30
31	.5767	.5934	.6100	.6266	.6432	.6597	.6761	.6925	.7089	.7251	.7414	31
32	.5769	.5936	.6103	.6269	.6434	.6599	.6764	.6928	.7091	.7254	.7417	32
33	.5772	.5939	.6106	.6272	.6437	.6602	.6767	.6931	.7094	.7257	.7419	33
34	.5775	.5942	.6108	.6274	.6440	.6605	.6769	.6933	.7097	.7260	.7422	34
35	.5778	.5945	.6111	.6277	.6443	.6608	.6772	.6936	.7099	.7262	.7425	35
36	.5781	.5947	.6114	.6280	.6445	.6610	.6775	.6939	.7102	.7265	.7427	36
37	.5783	.5950	.6117	.6283	.6448	.6613	.6777	.6941	.7105	.7268	.7430	37
38	.5786	.5953	.6119	.6285	.6451	.6616	.6780	.6944	.7108	.7270	.7433	38
39	.5789	.5956	.6122	.6288	.6454	.6619	.6783	.6947	.7110	.7273	.7435	39
40	.5792	.5959	.6125	.6291	.6456	.6621	.6786	.6950	.7113	.7276	.7438	40
41	.5795	.5961	.6128	.6294	.6459	.6624	.6788	.6952	.7116	.7279	.7441	41
42	.5797	.5964	.6130	.6296	.6462	.6627	.6791	.6955	.7118	.7281	.7443	42
43	.5800	.5967	.6133	.6299	.6465	.6630	.6794	.6958	.7121	.7284	.7446	43
44	.5803	.5970	.6136	.6302	.6467	.6632	.6797	.6961	.7124	.7287	.7449	44
45	.5806	.5972	.6139	.6305	.6470	.6635	.6799	.6963	.7127	.7289	.7452	45
46	.5808	.5975	.6142	.6307	.6473	.6638	.6802	.6966	.7129	.7292	.7454	46
47	.5811	.5978	.6144	.6310	.6476	.6640	.6805	.6969	.7132	.7295	.7457	47
48	.5814	.5981	.6147	.6313	.6478	.6643	.6808	.6971	.7135	.7298	.7460	48
49	.5817	.5984	.6150	.6316	.6481	.6646	.6810	.6974	.7137	.7300	.7462	49
50	.5820	.5986	.6153	.6318	.6484	.6649	.6813	.6977	.7140	.7303	.7465	50
51	.5822	.5989	.6155	.6321	.6487	.6651	.6816	.6980	.7143	.7306	.7468	51
52	.5825	.5992	.6158	.6324	.6489	.6654	.6819	.6982	.7146	.7308	.7471	52
53	.5828	.5995	.6161	.6327	.6492	.6657	.6821	.6985	.7148	.7311	.7473	53
54	.5831	.5997	.6164	.6330	.6495	.6660	.6824	.6988	.7151	.7314	.7476	54
55	.5834	.6000	.6166	.6332	.6498	.6662	.6827	.6991	.7154	.7316	.7479	55
56	.5836	.6003	.6169	.6335	.6500	.6665	.6829	.6993	.7156	.7319	.7481	56
57	.5839	.6006	.6172	.6338	.6503	.6668	.6832	.6996	.7159	.7322	.7484	57
58	.5842	.6009	.6175	.6341	.6506	.6671	.6835	.6999	.7162	.7325	.7487	58
59	.5845	.6011	.6178	.6343	.6509	.6673	.6838	.7001	.7165	.7327	.7489	59
30	.5847	.6014	.6180	.6346	.6511	.6676	.6840	.7004	.7167	.7330	.7492	60

4°	45°	46°	47°	48°	49°	50°	51°	52°	53°	54°	M.
492	.7654	.7815	.7975	.8135	.8294	.8452	.8610	.8767	.8924	.9080	0′
495	.7656	.7817	.7978	.8137	.8297	.8455	.8613	.8770	.8927	.9082	1
498	.7659	.7820	.7980	.8140	.8299	.8458	.8615	.8773	.8929	.9085	2
00	.7662	.7823	.7983	.8143	.8302	.8460	.8618	.8775	.8932	.9088	3
03	.7664	.7825	.7986	.8145	.8304	.8463	.8621	.8778	.8934	.9090	4
06	.7667	.7828	.7988	.8148	.8307	.8466	.8623	.8780	.8937	.9093	5
08	.7670	.7831	.7991	.8151	.8310	.8468	.8626	.8783	.8940	.9095	6
11	.7672	.7833	.7994	.8153	.8312	.8471	.8629	.8786	.8942	.9098	7
14	.7675	.7836	.7996	.8156	.8315	.8473	.8631	.8788	.8945	.9101	8
16	.7678	.7839	.7999	.8159	.8318	.8476	.8634	.8791	.8947	.9103	9
19	.7681	.7841	.8002	.8161	.8320	.8479	.8636	.8794	.8950	.9106	10
22	.7683	.7844	.8004	.8164	.8323	.8481	.8639	.8796	.8953	.9108	11
24	.7686	.7847	.8007	.8167	.8326	.8484	.8642	.8799	.8955	.9111	12
27	.7689	.7849	.8010	.8169	.8328	.8487	.8644	.8801	.8958	.9113	13
30	.7691	.7852	.8012	.8172	.8331	.8489	.8647	.8804	.8960	.9116	14
33	.7694	.7855	.8015	.8175	.8334	.8492	.8650	.8807	.8963	.9119	15
35	.7697	.7857	.8018	.8177	.8336	.8495	.8652	.8809	.8966	.9121	16
38	.7699	.7860	.8020	.8180	.8339	.8497	.8655	.8812	.8968	.9124	17
41	.7702	.7863	.8023	.8183	.8341	.8500	.8657	.8814	.8971	.9126	18
43	.7705	.7865	.8026	.8185	.8344	.8502	.8660	.8817	.8973	.9129	19
46	.7707	.7868	.8028	.8188	.8347	.8505	.8663	.8820	.8976	.9132	20
49	.7710	.7871	.8031	.8190	.8349	.8508	.8665	.8822	.8979	.9134	21
51	.7713	.7873	.8034	.8193	.8352	.8510	.8668	.8825	.8981	.9137	22
54	.7715	.7876	.8036	.8196	.8355	.8513	.8671	.8828	.8984	.9139	23
57	.7718	.7879	.8039	.8198	.8357	.8516	.8673	.8830	.8986	.9142	24
60	.7721	.7882	.8042	.8201	.8360	.8518	.8676	.8833	.8989	.9145	25
62	.7723	.7884	.8044	.8204	.8363	.8521	.8678	.8835	.8992	.9147	26
65	.7726	.7887	.8047	.8206	.8365	.8523	.8681	.8838	.8994	.9150	27
68	.7729	.7890	.8050	.8209	.8368	.8526	.8684	.8841	.8997	.9152	28
70	.7731	.7892	.8052	.8212	.8371	.8529	.8686	.8843	.8999	.9155	29
73	.7734	.7895	.8055	.8214	.8373	.8531	.8689	.8846	.9002	.9157	30
76	.7737	.7898	.8058	.8217	.8376	.8534	.8692	.8848	.9005	.9160	31
78	.7740	.7900	.8060	.8220	.8378	.8537	.8694	.8851	.9007	.9163	32
81	.7742	.7903	.8063	.8222	.8381	.8539	.8697	.8854	.9010	.9165	33
84	.7745	.7906	.8066	.8225	.8384	.8542	.8699	.8856	.9012	.9168	34
86	.7748	.7908	.8068	.8228	.8386	.8545	.8702	.8859	.9015	.9170	35
89	.7750	.7911	.8071	.8230	.8389	.8547	.8705	.8861	.9018	.9173	36
92	.7753	.7914	.8074	.8233	.8392	.8550	.8707	.8864	.9020	.9176	37
95	.7756	.7916	.8076	.8236	.8394	.8552	.8710	.8867	.9023	.9178	38
97	.7758	.7919	.8079	.8238	.8397	.8555	.8712	.8869	.9025	.9181	39
00	.7761	.7922	.8082	.8241	.8400	.8558	.8715	.8872	.9028	.9183	40
03	.7764	.7924	.8084	.8244	.8402	.8560	.8718	.8874	.9031	.9186	41
05	.7766	.7927	.8087	.8246	.8405	.8563	.8720	.8877	.9033	.9188	42
08	.7769	.7930	.8090	.8249	.8408	.8566	.8723	.8880	.9036	.9191	43
11	.7772	.7932	.8092	.8251	.8410	.8568	.8726	.8882	.9038	.9194	44
13	.7774	.7935	.8095	.8254	.8413	.8571	.8728	.8885	.9041	.9196	45
16	.7777	.7938	.8098	.8257	.8415	.8573	.8731	.8887	.9044	.9199	46
19	.7780	.7940	.8100	.8259	.8418	.8576	.8734	.8890	.9046	.9201	47
21	.7782	.7943	.8103	.8262	.8421	.8579	.8736	.8893	.9049	.9204	48
24	.7785	.7946	.8105	.8265	.8423	.8581	.8739	.8895	.9051	.9207	49
27	.7788	.7948	.8108	.8267	.8426	.8584	.8741	.8898	.9054	.9209	50
29	.7791	.7951	.8111	.8270	.8429	.8587	.8744	.8900	.9056	.9212	51
32	.7793	.7954	.8113	.8273	.8431	.8589	.8747	.8903	.9059	.9214	52
35	.7796	7956	.8116	.8275	.8434	.8592	.8749	.8906	.9062	.9217	53
38	.7799	.7959	.8119	.8278	.8437	.8594	.8752	.8908	.9064	.9219	54
40	.7801	.7962	.8121	.8281	.8439	.8597	.8754	.8911	.9067	.9222	55
43	.7804	.7964	.8124	.8283	.8442	.8600	.8757	.8914	.9069	.9225	56
46	.7807	.7967	.8127	.8286	.8444	.8602	.8760	.8916	.9072	.9227	57
48	.7809	.7970	.8129	.8289	.8447	.8605	.8762	.8919	.9075	.9230	58
51	.7812	.7972	.8132	.8291	.8450	.8608	.8765	.8921	.9077	.9232	59
54	.7815	.7975	.8135	.8294	.8452	.8610	.8767	.8924	.9080	.9235	60

Table of Chords; Radius = 1.0000 (*continued*).

M.	55°	56°	57°	58°	59°	60°	61°	62°	63°	64°	M.
0′	.9235	.9389	.9543	.9696	.9848	1.0000	1.0151	1.0301	1.0450	1.0598	0′
1	.9238	.9392	.9546	.9699	.9851	1.0003	1.0153	1.0303	1.0452	1.0601	1
2	.9240	.9395	.9548	.9701	.9854	1.0005	1.0156	1.0306	1.0455	1.0603	2
3	.9243	.9397	.9551	.9704	.9856	1.0008	1.0158	1.0308	1.0457	1.0606	3
4	.9245	.9400	.9553	.9706	.9859	1.0010	1.0161	1.0311	1.0460	1.0608	4
5	.9248	.9402	.9556	.9709	.9861	1.0013	1.0163	1.0313	1.0462	1.0611	5
6	.9250	.9405	.9559	.9711	.9864	1.0015	1.0166	1.0316	1.0465	1.0613	6
7	.9253	.9407	.9561	.9714	.9866	1.0018	1.0168	1.0318	1.0467	1.0616	7
8	.9256	.9410	.9564	.9717	.9869	1.0020	1.0171	1.0321	1.0470	1.0618	8
9	.9258	.9413	.9566	.9719	.9871	1.0023	1.0173	1.0323	1.0472	1.0621	9
10	.9261	.9415	.9569	.9722	.9874	1.0025	1.0176	1.0326	1.0475	1.0623	10
11	.9263	.9418	.9571	.9724	.9876	1.0028	1.0178	1.0328	1.0477	1.0626	11
12	.9266	.9420	.9574	.9727	.9879	1.0030	1.0181	1.0331	1.0480	1.0628	12
13	.9268	.9423	.9576	.9729	.9881	1.0033	1.0183	1.0333	1.0482	1.0630	13
14	.9271	.9425	.9579	.9732	.9884	1.0035	1.0186	1.0336	1.0485	1.0633	14
15	.9274	.9428	.9581	.9734	.9886	1.0038	1.0188	1.0338	1.0487	1.0635	15
16	.9276	.9430	.9584	.9737	.9889	1.0040	1.0191	1.0341	1.0490	1.0638	16
17	.9279	.9433	.9587	.9739	.9891	1.0043	1.0193	1.0343	1.0492	1.0640	17
18	.9281	.9436	.9589	.9742	.9894	1.0045	1.0196	1.0346	1.0495	1.0643	18
19	.9284	.9438	.9592	.9744	.9897	1.0048	1.0198	1.0348	1.0497	1.0645	19
20	.9287	.9441	.9594	.9747	.9899	1.0050	1.0201	1.0351	1.0500	1.0648	20
21	.9289	.9443	.9597	.9750	.9902	1.0053	1.0203	1.0353	1.0502	1.0650	21
22	.9292	.9446	.9599	.9752	.9904	1.0055	1.0206	1.0356	1.0504	1.0653	22
23	.9294	.9448	.9602	.9755	.9907	1.0058	1.0208	1.0358	1.0507	1.0655	23
24	.9297	.9451	.9604	.9757	.9909	1.0060	1.0211	1.0361	1.0509	1.0658	24
25	.9299	.9454	.9607	.9760	.9912	1.0063	1.0213	1.0363	1.0512	1.0660	25
26	.9302	.9456	.9610	.9762	.9914	1.0065	1.0216	1.0366	1.0514	1.0662	26
27	.9305	.9459	.9612	.9765	.9917	1.0068	1.0218	1.0368	1.0517	1.0665	27
28	.9307	.9461	.9615	.9767	.9919	1.0070	1.0221	1.0370	1.0519	1.0667	28
29	.9310	.9464	.9617	.9770	.9922	1.0073	1.0223	1.0373	1.0522	1.0670	29
30	.9312	.9466	.9620	.9772	.9924	1.0075	1.0226	1.0375	1.0524	1.0672	30
31	.9315	.9469	.9622	.9775	.9927	1.0078	1.0228	1.0378	1.0527	1.0675	31
32	.9317	.9472	.9625	.9778	.9929	1.0080	1.0231	1.0380	1.0529	1.0677	32
33	.9320	.9474	.9627	.9780	.9932	1.0083	1.0233	1.0383	1.0532	1.0680	33
34	.9323	.9477	.9630	.9783	.9934	1.0086	1.0236	1.0385	1.0534	1.0682	34
35	.9325	.9479	.9633	.9785	.9937	1.0088	1.0238	1.0388	1.0537	1.0685	35
36	.9328	.9482	.9635	.9788	.9939	1.0091	1.0241	1.0390	1.0539	1.0687	36
37	.9330	.9484	.9638	.9790	.9942	1.0093	1.0243	1.0393	1.0542	1.0690	37
38	.9333	.9487	.9640	.9793	.9945	1.0096	1.0246	1.0395	1.0544	1.0692	38
39	.9335	.9489	.9643	.9795	.9947	1.0098	1.0248	1.0398	1.0547	1.0694	39
40	.9338	.9492	.9645	.9798	.9950	1.0101	1.0251	1.0400	1.0549	1.0697	40
41	.9341	.9495	.9648	.9800	.9952	1.0103	1.0253	1.0403	1.0551	1.0699	41
42	.9343	.9497	.9650	.9803	.9955	1.0106	1.0256	1.0405	1.0554	1.0702	42
43	.9346	.9500	.9653	.9805	.9957	1.0108	1.0258	1.0408	1.0556	1.0704	43
44	.9348	.9502	.9655	.9808	.9960	1.0111	1.0261	1.0410	1.0559	1.0707	44
45	.9351	.9505	.9658	.9810	.9962	1.0113	1.0263	1.0413	1.0561	1.0709	45
46	.9353	.9507	.9661	.9813	.9965	1.0116	1.0266	1.0415	1.0564	1.0712	46
47	.9356	.9510	.9663	.9816	.9967	1.0118	1.0268	1.0418	1.0566	1.0714	47
48	.9359	.9512	.9666	.9818	.9970	1.0121	1.0271	1.0420	1.0569	1.0717	48
49	.9361	.9515	.9668	.9821	.9972	1.0123	1.0273	1.0423	1.0571	1.0719	49
50	.9364	.9518	.9671	.9823	.9975	1.0126	1.0276	1.0425	1.0574	1.0721	50
51	.9366	.9520	.9673	.9826	.9977	1.0128	1.0278	1.0428	1.0576	1.0724	51
52	.9369	.9523	.9676	.9828	.9980	1.0131	1.0281	1.0430	1.0579	1.0726	52
53	.9371	.9525	.9678	.9831	.9982	1.0133	1.0283	1.0433	1.0581	1.0729	53
54	.9374	.9528	.9681	.9833	.9985	1.0136	1.0286	1.0435	1.0584	1.0731	54
55	.9377	.9530	.9683	.9836	.9987	1.0138	1.0288	1.0438	1.0586	1.0734	55
56	.9379	.9533	.9686	.9838	.9990	1.0141	1.0291	1.0440	1.0589	1.0736	56
57	.9382	.9536	.9689	.9841	.9992	1.0143	1.0293	1.0443	1.0591	1.0739	57
58	.9384	.9538	.9691	.9843	.9995	1.0146	1.0296	1.0445	1.0593	1.0741	58
59	.9387	.9541	.9694	.9846	.9998	1.0148	1.0298	1.0447	1.0596	1.0744	59
60	.9389	.9543	.9696	.9848	1.0000	1.0151	1.0301	1.0450	1.0598	1.0746	60

Table of Chords; Radius = 1.0000 (*continued*).

M.	65°	66°	67°	68°	69°	70°	71°	72°	73°	M.
0′	1.0746	1.0893	1.1039	1.1184	1.1328	1.1472	1.1614	1.1756	1.1896	0′
1	1.0748	1.0895	1.1041	1.1186	1.1331	1.1474	1.1616	1.1758	1.1899	1
2	1.0751	1.0898	1.1044	1.1189	1.1333	1.1476	1.1619	1.1760	1.1901	2
3	1.0753	1.0900	1.1046	1.1191	1.1335	1.1479	1.1621	1.1763	1.1903	3
4	1.0756	1.0903	1.1048	1.1194	1.1338	1.1481	1.1624	1.1765	1.1906	4
5	1.0758	1.0905	1.1051	1.1196	1.1340	1.1483	1.1626	1.1767	1.1908	5
6	1.0761	1.0907	1.1053	1.1198	1.1342	1.1486	1.1628	1.1770	1.1910	6
7	1.0763	1.0910	1.1056	1.1201	1.1345	1.1488	1.1631	1.1772	1.1913	7
8	1.0766	1.0912	1.1058	1.1203	1.1347	1.1491	1.1633	1.1775	1.1915	8
9	1.0768	1.0915	1.1061	1.1206	1.1350	1.1493	1.1635	1.1777	1.1917	9
10	1.0771	1.0917	1.1063	1.1208	1.1352	1.1495	1.1638	1.1779	1.1920	10
11	1.0773	1.0920	1.1065	1.1210	1.1354	1.1498	1.1640	1.1782	1.1922	11
12	1.0775	1.0922	1.1068	1.1213	1.1357	1.1500	1.1642	1.1784	1.1924	12
13	1.0778	1.0924	1.1070	1.1215	1.1359	1.1502	1.1645	1.1786	1.1927	13
14	1.0780	1.0927	1.1073	1.1218	1.1362	1.1505	1.1647	1.1789	1.1929	14
15	1.0783	1.0929	1.1075	1.1220	1.1364	1.1507	1.1650	1.1791	1.1931	15
16	1.0785	1.0932	1.1078	1.1222	1.1366	1.1510	1.1652	1.1793	1.1934	16
17	1.0788	1.0934	1.1080	1.1225	1.1369	1.1512	1.1654	1.1796	1.1936	17
18	1.0790	1.0937	1.1082	1.1227	1.1371	1.1514	1.1657	1.1798	1.1938	18
19	1.0793	1.0939	1.1085	1.1230	1.1374	1.1517	1.1659	1.1800	1.1941	19
20	1.0795	1.0942	1.1087	1.1232	1.1376	1.1519	1.1661	1.1803	1.1943	20
21	1.0797	1.0944	1.1090	1.1234	1.1378	1.1522	1.1664	1.1805	1.1946	21
22	1.0800	1.0946	1.1092	1.1237	1.1381	1.1524	1.1666	1.1807	1.1948	22
23	1.0802	1.0949	1.1094	1.1239	1.1383	1.1526	1.1668	1.1810	1.1950	23
24	1.0805	1.0951	1.1097	1.1242	1.1386	1.1529	1.1671	1.1812	1.1952	24
25	1.0807	1.0954	1.1099	1.1244	1.1388	1.1531	1.1673	1.1814	1.1955	25
26	1.0810	1.0956	1.1102	1.1246	1.1390	1.1533	1.1676	1.1817	1.1957	26
27	1.0812	1.0959	1.1104	1.1249	1.1393	1.1536	1.1678	1.1819	1.1959	27
28	1.0815	1.0961	1.1107	1.1251	1.1395	1.1538	1.1680	1.1821	1.1962	28
29	1.0817	1.0963	1.1109	1.1254	1.1398	1.1541	1.1683	1.1824	1.1964	29
30	1.0820	1.0966	1.1111	1.1256	1.1400	1.1543	1.1685	1.1826	1.1966	30
31	1.0822	1.0968	1.1114	1.1258	1.1402	1.1545	1.1687	1.1829	1.1969	31
32	1.0824	1.0971	1.1116	1.1261	1.1405	1.1548	1.1690	1.1831	1.1971	32
33	1.0827	1.0973	1.1119	1.1263	1.1407	1.1550	1.1692	1.1833	1.1973	33
34	1.0829	1.0976	1.1121	1.1266	1.1409	1.1552	1.1694	1.1836	1.1976	34
35	1.0832	1.0978	1.1123	1.1268	1.1412	1.1555	1.1697	1.1838	1.1978	35
36	1.0834	1.0980	1.1126	1.1271	1.1414	1.1557	1.1699	1.1840	1.1980	36
37	1.0837	1.0983	1.1128	1.1273	1.1417	1.1560	1.1702	1.1843	1.1983	37
38	1.0839	1.0985	1.1131	1.1275	1.1419	1.1562	1.1704	1.1845	1.1985	38
39	1.0841	1.0988	1.1133	1.1278	1.1421	1.1564	1.1706	1.1847	1.1987	39
40	1.0844	1.0990	1.1136	1.1280	1.1424	1.1567	1.1709	1.1850	1.1990	40
41	1.0846	1.0993	1.1138	1.1283	1.1426	1.1569	1.1711	1.1852	1.1992	41
42	1.0849	1.0995	1.1140	1.1285	1.1429	1.1571	1.1713	1.1854	1.1994	42
43	1.0851	1.0997	1.1143	1.1287	1.1431	1.1574	1.1716	1.1857	1.1997	43
44	1.0854	1.1000	1.1145	1.1290	1.1433	1.1576	1.1718	1.1859	1.1999	44
45	1.0856	1.1002	1.1148	1.1292	1.1436	1.1579	1.1720	1.1861	1.2001	45
46	1.0859	1.1005	1.1150	1.1295	1.1438	1.1581	1.1723	1.1864	1.2004	46
47	1.0861	1.1007	1.1152	1.1297	1.1441	1.1583	1.1725	1.1866	1.2006	47
48	1.0863	1.1010	1.1155	1.1299	1.1443	1.1586	1.1727	1.1868	1.2008	48
49	1.0866	1.1012	1.1157	1.1302	1.1445	1.1588	1.1730	1.1871	1.2011	49
50	1.0868	1.1014	1.1160	1.1304	1.1448	1.1590	1.1732	1.1873	1.2013	50
51	1.0871	1.1017	1.1162	1.1307	1.1450	1.1593	1.1735	1.1875	1.2015	51
52	1.0873	1.1019	1.1165	1.1309	1.1452	1.1595	1.1737	1.1878	1.2018	52
53	1.0876	1.1022	1.1167	1.1311	1.1455	1.1598	1.1739	1.1880	1.2020	53
54	1.0878	1.1024	1.1169	1.1314	1.1457	1.1600	1.1742	1.1882	1.2022	54
55	1.0881	1.1027	1.1172	1.1316	1.1460	1.1602	1.1744	1.1885	1.2025	55
56	1.0883	1.1029	1.1174	1.1319	1.1462	1.1605	1.1746	1.1887	1.2027	56
57	1.0885	1.1031	1.1177	1.1321	1.1464	1.1607	1.1749	1.1889	1.2029	57
58	1.0888	1.1034	1.1179	1.1323	1.1467	1.1609	1.1751	1.1892	1.2032	58
59	1.0890	1.1036	1.1181	1.1326	1.1469	1.1612	1.1753	1.1894	1.2034	59
60	1.0893	1.1039	1.1184	1.1328	1.1472	1.1614	1.1756	1.1896	1.2036	60

Table of Chords; Radius = 1.0000 (*continued*).

M.	74°	75°	76°	77°	78°	79°	80°	81°	82°	M.
0′	1.2036	1.2175	1.2313	1.2450	1.2586	1.2722	1.2856	1.2989	1.3121	0′
1	1.2039	1.2178	1.2316	1.2453	1.2589	1.2724	1.2858	1.2991	1.3123	1
2	1.2041	1.2180	1.2318	1.2455	1.2591	1.2726	1.2860	1.2993	1.3126	2
3	1.2043	1.2182	1.2320	1.2457	1.2593	1.2728	1.2862	1.2996	1.3128	3
4	1.2046	1.2184	1.2322	1.2459	1.2595	1.2731	1.2865	1.2998	1.3130	4
5	1.2048	1.2187	1.2325	1.2462	1.2598	1.2733	1.2867	1.3000	1.3132	5
6	1.2050	1.2189	1.2327	1.2464	1.2600	1.2735	1.2869	1.3002	1.3134	6
7	1.2053	1.2191	1.2329	1.2466	1.2602	1.2737	1.2871	1.3004	1.3137	7
8	1.2055	1.2194	1.2332	1.2468	1.2604	1.2740	1.2874	1.3007	1.3139	8
9	1.2057	1.2196	1.2334	1.2471	1.2607	1.2742	1.2876	1.3009	1.3141	9
10	1.2060	1.2198	1.2336	1.2473	1.2609	1.2744	1.2878	1.3011	1.3143	10
11	1.2062	1.2201	1.2338	1.2475	1.2611	1.2746	1.2880	1.3013	1.3145	11
12	1.2064	1.2203	1.2341	1.2478	1.2614	1.2748	1.2882	1.3015	1.3147	12
13	1.2066	1.2205	1.2343	1.2480	1.2616	1.2751	1.2885	1.3018	1.3150	13
14	1.2069	1.2208	1.2345	1.2482	1.2618	1.2753	1.2887	1.3020	1.3152	14
15	1.2071	1.2210	1.2348	1.2484	1.2620	1.2755	1.2889	1.3022	1.3154	15
16	1.2073	1.2212	1.2350	1.2487	1.2623	1.2757	1.2891	1.3024	1.3156	16
17	1.2076	1.2214	1.2352	1.2489	1.2625	1.2760	1.2894	1.3027	1.3158	17
18	1.2078	1.2217	1.2354	1.2491	1.2627	1.2762	1.2896	1.3029	1.3161	18
19	1.2080	1.2219	1.2357	1.2493	1.2629	1.2764	1.2898	1.3031	1.3163	19
20	1.2083	1.2221	1.2359	1.2496	1.2632	1.2766	1.2900	1.3033	1.3165	20
21	1.2085	1.2224	1.2361	1.2498	1.2634	1.2769	1.2903	1.3035	1.3167	21
22	1.2087	1.2226	1.2364	1.2500	1.2636	1.2771	1.2905	1.3038	1.3169	22
23	1.2090	1.2228	1.2366	1.2503	1.2638	1.2773	1.2907	1.3040	1.3172	23
24	1.2092	1.2231	1.2368	1.2505	1.2641	1.2775	1.2909	1.3042	1.3174	24
25	1.2094	1.2233	1.2370	1.2507	1.2643	1.2778	1.2911	1.3044	1.3176	25
26	1.2097	1.2235	1.2373	1.2509	1.2645	1.2780	1.2914	1.3046	1.3178	26
27	1.2099	1.2237	1.2375	1.2512	1.2648	1.2782	1.2916	1.3049	1.3180	27
28	1.2101	1.2240	1.2377	1.2514	1.2650	1.2784	1.2918	1.3051	1.3183	28
29	1.2104	1.2242	1.2380	1.2516	1.2652	1.2787	1.2920	1.3053	1.3185	29
30	1.2106	1.2244	1.2382	1.2518	1.2654	1.2789	1.2922	1.3055	1.3187	30
31	1.2108	1.2247	1.2384	1.2521	1.2656	1.2791	1.2925	1.3057	1.3189	31
32	1.2111	1.2249	1.2386	1.2523	1.2659	1.2793	1.2927	1.3060	1.3191	32
33	1.2113	1.2251	1.2389	1.2525	1.2661	1.2795	1.2929	1.3062	1.3193	33
34	1.2115	1.2254	1.2391	1.2528	1.2663	1.2798	1.2931	1.3064	1.3196	34
35	1.2117	1.2256	1.2393	1.2530	1.2665	1.2800	1.2934	1.3066	1.3198	35
36	1.2120	1.2258	1.2396	1.2532	1.2668	1.2802	1.2936	1.3068	1.3200	36
37	1.2122	1.2260	1.2398	1.2534	1.2670	1.2804	1.2938	1.3071	1.3202	37
38	1.2124	1.2263	1.2400	1.2537	1.2672	1.2807	1.2940	1.3073	1.3204	38
39	1.2127	1.2265	1.2402	1.2539	1.2674	1.2809	1.2942	1.3075	1.3207	39
40	1.2129	1.2267	1.2405	1.2541	1.2677	1.2811	1.2945	1.3077	1.3209	40
41	1.2131	1.2270	1.2407	1.2543	1.2679	1.2813	1.2947	1.3079	1.3211	41
42	1.2134	1.2272	1.2409	1.2546	1.2681	1.2816	1.2949	1.3082	1.3213	42
43	1.2136	1.2274	1.2412	1.2548	1.2683	1.2818	1.2951	1.3084	1.3215	43
44	1.2138	1.2277	1.2414	1.2550	1.2686	1.2820	1.2954	1.3086	1.3218	44
45	1.2141	1.2279	1.2416	1.2552	1.2688	1.2822	1.2956	1.3088	1.3220	45
46	1.2143	1.2281	1.2418	1.2555	1.2690	1.2825	1.2958	1.3090	1.3222	46
47	1.2145	1.2283	1.2421	1.2557	1.2692	1.2827	1.2960	1.3093	1.3224	47
48	1.2148	1.2286	1.2423	1.2559	1.2695	1.2829	1.2962	1.3095	1.3226	48
49	1.2150	1.2288	1.2425	1.2562	1.2697	1.2831	1.2965	1.3097	1.3228	49
50	1.2152	1.2290	1.2428	1.2564	1.2699	1.2833	1.2967	1.3099	1.3231	50
51	1.2154	1.2293	1.2430	1.2566	1.2701	1.2836	1.2969	1.3101	1.3233	51
52	1.2157	1.2295	1.2432	1.2568	1.2704	1.2838	1.2971	1.3104	1.3235	52
53	1.2159	1.2297	1.2434	1.2571	1.2706	1.2840	1.2973	1.3106	1.3237	53
54	1.2161	1.2299	1.2437	1.2573	1.2708	1.2842	1.2976	1.3108	1.3239	54
55	1.2164	1.2302	1.2439	1.2575	1.2710	1.2845	1.2978	1.3110	1.3242	55
56	1.2166	1.2304	1.2441	1.2577	1.2713	1.2847	1.2980	1.3112	1.3244	56
57	1.2168	1.2306	1.2443	1.2580	1.2715	1.2849	1.2982	1.3115	1.3246	57
58	1.2171	1.2309	1.2446	1.2582	1.2717	1.2851	1.2985	1.3117	1.3248	58
59	1.2173	1.2311	1.2448	1.2584	1.2719	1.2854	1.2987	1.3119	1.3250	59
60	1.2175	1.2313	1.2450	1.2586	1.2722	1.2856	1.2989	1.3121	1.3252	60

Lengths and Bevels of Hip and Jack Rafters.

The lines *ab* and *bc* in Fig. 89 represent the walls at the angle of a building; *be* is the seat of the hip-rafter, and *gf* of a jack-rafter. Draw *eh* at right angles to *be*, and make it equal to the rise of the roof; join *b* and *h*, and *hb* will be the length of the hip-rafter. Through *e* draw *di* at right angles to *bc*. Upon *b*, with the radius *bh*, describe the arc *hi*, cutting *di* in *i*. Join *b* and *i*, and extend *gf*

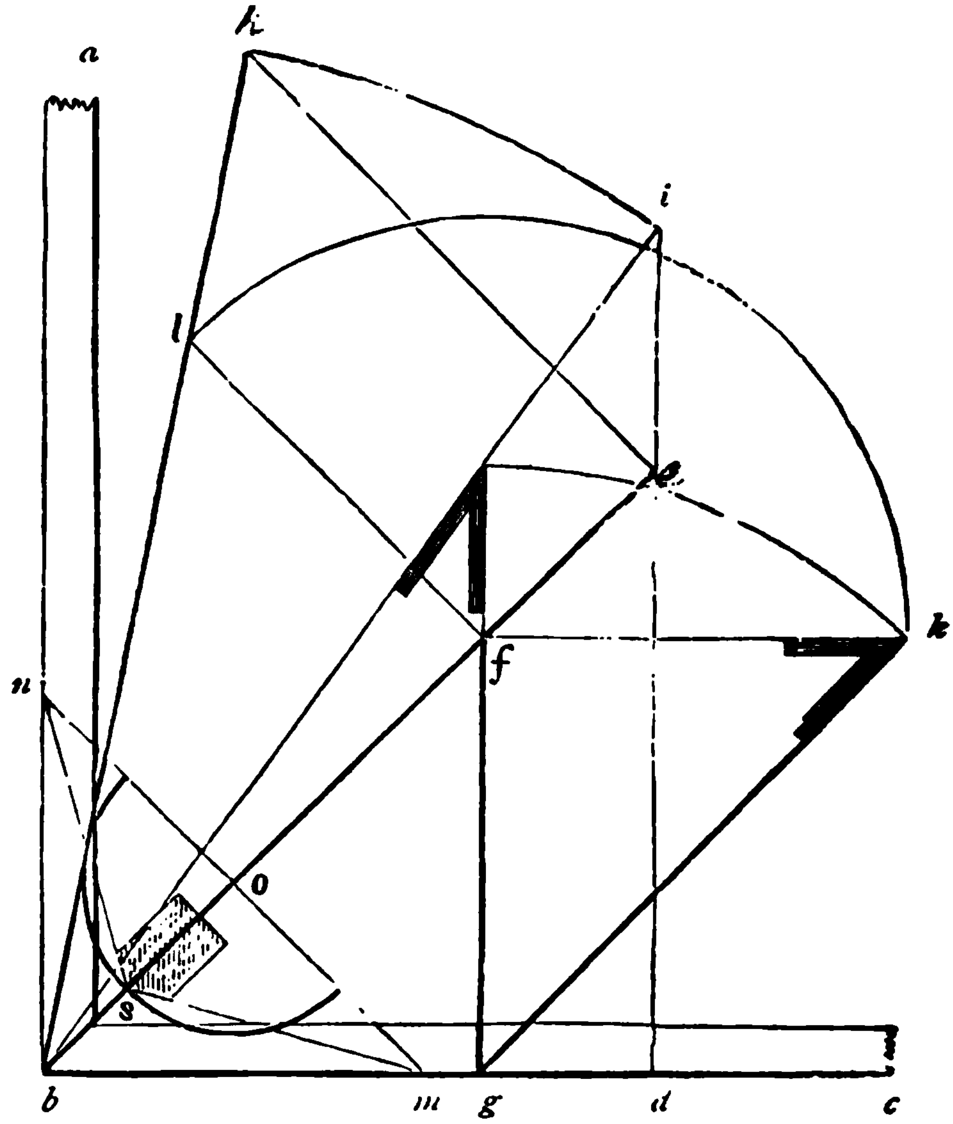


Fig. 89.

to meet *bi* in *j*; then *gj* will be the length of the **jack-rafter. The** length of each jack-rafter is found in the same **manner, — by ex**tending its seat to cut the line *bi*. From *f* draw ***fk* at right angles** to *fg*, also *fl* at right angles to *be*. Make *fk* equal **to *fl* by the arc** *lk*, or make *gk* equal to *gj* by the arc *jk*; then the angle **at *j* will be** the *top bevel* of the jack-rafters, and the one at *k* the ***down bevel.***

Backing of the hip-rafter. At any convenient place in *be* (**Fig.** 89), as *o*, draw *mn* at right angles to *be*. From *o* **describe a circle,** tangent to *bh*, cutting *be* in *s*. Join *m* and *s* **and *n* and *s*; then** these lines will form at *s* the proper angle for **bevelling the top of** the hip-rafter.

TRIGONOMETRY.

ot the purpose of the author to teach the use of trigonom-
what it is; but, for the benefit of those readers who have
acquired a knowledge of this science, the following con-
formulas, and tables of natural sines and tangents, have
erted. To those who know how to apply these trigono-
unctions, they will often be found of great convenience
ty.

tables are taken from Searle's "Field Engineering," John
Sons, publishers, by permission.

Trigonometric Functions.

Let A (Fig. 107) = angle BAC = arc BF, and let the radius $AF = AB = AH = 1$.

We then have

$\sin A$	$= BC$
$\cos A$	$= AC$
$\tan A$	$= DF$
$\cot A$	$= HG$
$\sec A$	$= AD$
$\operatorname{cosec} A$	$= AG$
$\operatorname{versin} A$	$= CF = BE$
$\operatorname{covers} A$	$= BK = HL$
$\operatorname{exsec} A$	$= BD$
$\operatorname{coexsec} A$	$= BG$
$\operatorname{chord} A$	$= BF$
$\operatorname{chord} 2A$	$= BI = 2BC$

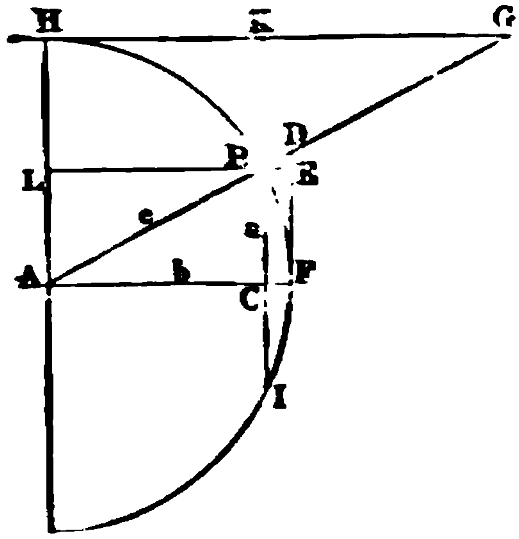


Fig. 107.

In the right-angled triangle ABC (Fig. 107)

Let $AB = c$, $AC = b$, and $BC = a$.

We then have:

1. $\sin A = \dfrac{a}{c} = \cos B$
2. $\cos A = \dfrac{b}{c} = \sin B$
3. $\tan A = \dfrac{a}{b} = \cot B$
4. $\cot A = \dfrac{b}{a} = \tan B$
5. $\sec A = \dfrac{c}{b} = \operatorname{cosec} B$
6. $\operatorname{cosec} A = \dfrac{c}{a} = \sec B$
7. $\operatorname{vers} A = \dfrac{c-b}{c} = \operatorname{covers} B$
8. $\operatorname{exsec} A = \dfrac{c-b}{b} = \operatorname{coexsec} B$
9. $\operatorname{covers} A = \dfrac{c-a}{c} = \operatorname{versin} B$
10. $\operatorname{coexsec} A = \dfrac{c-a}{a} = \operatorname{exsec} B$
11. $a = c \sin A = b \tan A$
12. $b = c \cos A = a \cot A$
13. $c = \dfrac{a}{\sin A} = \dfrac{b}{\cos A}$
14. $a = c \cos B = b \cot B$
15. $b = c \sin B = a \tan B$
16. $c = \dfrac{a}{\cos B} = \dfrac{b}{\sin B}$
17. $a = \sqrt{(c+b)(c-b)}$
18. $b = \sqrt{(c+a)(c-a)}$
19. $c = \sqrt{a^2 + b^2}$
20. $C = 90° = A + B$
21. Area $= \dfrac{ab}{2}$.

SOLUTION OF OBLIQUE TRIANGLES.

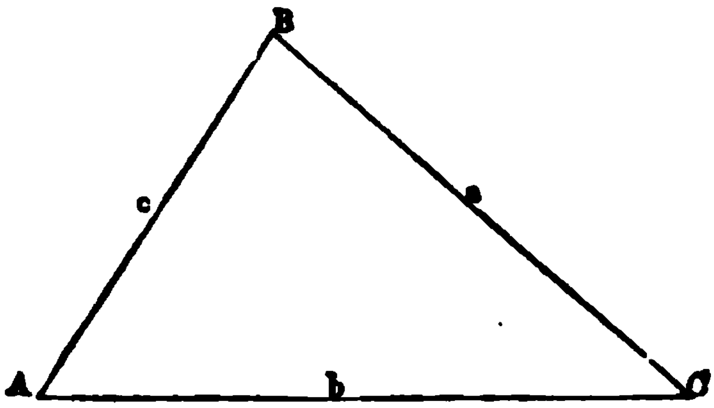


FIG. 108.

	GIVEN.	SOUGHT.	FORMULÆ.
22	A, B, a	C, b, c	$C = 180° - (A + B)$, $b = \frac{a}{\sin A} \cdot \sin B$, $c = \frac{a}{\sin A} \sin (A + B)$
23	A, a, b	B, C, c	$\sin B = \frac{\sin A}{a} \cdot b$, $C = 180° - (A + B)$, $c = \frac{a}{\sin A} \cdot \sin C$.
24	C, a, b	$\frac{1}{2}(A + B)$	$\frac{1}{2}(A + B) = 90° - \frac{1}{2} C$
25		$\frac{1}{2}(A - B)$	$\tan \frac{1}{2}(A - B) = \frac{a - b}{a + b} \tan \frac{1}{2}(A + B)$
26		A, B	$A = \frac{1}{2}(A + B) + \frac{1}{2}(A - B)$, $B = \frac{1}{2}(A + B) - \frac{1}{2}(A - B)$
27		c	$c = (a + b) \frac{\cos \frac{1}{2}(A + B)}{\cos \frac{1}{2}(A - B)} = (a - b) \frac{\sin \frac{1}{2}(A + B)}{\sin \frac{1}{2}(A - B)}$
28		area	$K = \frac{1}{2} a b \sin C$.
29	a, b, c	A	Let $s = \frac{1}{2}(a + b + c)$; $\sin \frac{1}{2} A = \sqrt{\frac{(s-b)(s-c)}{bc}}$
30			$\cos \frac{1}{2} A = \sqrt{\frac{s(s-a)}{bc}}$; $\tan \frac{1}{2} A = \sqrt{\frac{(s-b)(s-c)}{s(s-a)}}$
31			$\sin A = \frac{2\sqrt{s(s-a)(s-b)(s-c)}}{bc}$; $\text{vers } A = \frac{2(s-b)(s-c)}{bc}$
32		area	$K = \sqrt{s(s-a)(s-b)(s-c)}$
33	A, B, C, a	area	$K = \frac{a^2 \sin B \cdot \sin C}{2 \sin A}$

	GENERAL FORMULÆ.
34	$\sin A = \dfrac{1}{\operatorname{cosec} A} = \sqrt{1 - \cos^2 A} = \tan A \cos A$
35	$\sin A = 2 \sin \tfrac{1}{2} A \cos \tfrac{1}{2} A = \operatorname{vers} A \cot \tfrac{1}{2} A$
36	$\sin A = \sqrt{\tfrac{1}{2} \operatorname{vers} 2A} = \sqrt{\tfrac{1}{2}(1 - \cos 2A)}$
37	$\cos A = \dfrac{1}{\sec A} = \sqrt{1 - \sin^2 A} = \cot A \sin A$
38	$\cos A = 1 - \operatorname{vers} A = 2 \cos^2 \tfrac{1}{2} A - 1 = 1 - 2 \sin^2 \tfrac{1}{2} A$
39	$\cos A = \cos^2 \tfrac{1}{2} A - \sin^2 \tfrac{1}{2} A = \sqrt{\tfrac{1}{2} + \tfrac{1}{2} \cos 2A}$
40	$\tan A = \dfrac{1}{\cot A} = \dfrac{\sin A}{\cos A} = \sqrt{\sec^2 A - 1}$
41	$\tan A = \sqrt{\dfrac{1}{\cos^2 A} - 1} = \dfrac{\sqrt{1 - \cos^2 A}}{\cos A} = \dfrac{\sin 2A}{1 + \cos 2A}$
42	$\tan A = \dfrac{1 - \cos 2A}{\sin 2A} = \dfrac{\operatorname{vers} 2A}{\sin 2A} = \operatorname{exsec} A \cot \tfrac{1}{2} A$
43	$\cot A = \dfrac{1}{\tan A} = \dfrac{\cos A}{\sin A} = \sqrt{\operatorname{cosec}^2 A - 1}$
44	$\cot A = \dfrac{\sin 2A}{1 - \cos 2A} = \dfrac{\sin 2A}{\operatorname{vers} 2A} = \dfrac{1 + \cos 2A}{\sin 2A}$
45	$\cot A = \dfrac{\tan \tfrac{1}{2} A}{\operatorname{exsec} A}$
46	$\operatorname{vers} A = 1 - \cos A = \sin A \tan \tfrac{1}{2} A = 2 \sin^2 \tfrac{1}{2} A$
47	$\operatorname{vers} A = \operatorname{exsec} A \cos A$
48	$\operatorname{exsec} A = \sec A - 1 = \tan A \tan \tfrac{1}{2} A = \dfrac{\operatorname{vers} A}{\cos A}$
49	$\sin \tfrac{1}{2} A = \sqrt{\dfrac{1 - \cos A}{2}} = \sqrt{\dfrac{\operatorname{vers} A}{2}}$
50	$\sin 2A = 2 \sin A \cos A$
51	$\cos \tfrac{1}{2} A = \sqrt{\dfrac{1 + \cos A}{2}}$
52	$\cos 2A = 2 \cos^2 A - 1 = \cos^2 A - \sin^2 A = 1 - 2 \sin^2 A$

General Formulæ.

$$\tfrac{1}{2} A = \frac{\tan A}{1 + \sec A} = \operatorname{cosec} A - \cot A = \frac{1 - \cos A}{\sin A} = \sqrt{\frac{1 - \cos A}{1 + \cos A}}$$

$$2\, A = \frac{2 \tan A}{1 - \tan^2 A}$$

$$\tfrac{1}{2} A = \frac{\sin A}{\operatorname{vers} A} = \frac{1 + \cos A}{\sin A} = \frac{1}{\operatorname{cosec} A - \cot A}$$

$$2\, A = \frac{\cot^2 A - 1}{2 \cot A}$$

$$\tfrac{1}{2} A = \frac{\tfrac{1}{2} \operatorname{vers} A}{1 + \sqrt{1 - \tfrac{1}{2} \operatorname{vers} A}} = \frac{1 - \cos A}{2 + \sqrt{2 (1 + \cos A)}}$$

$$2\, A = 2 \sin^2 A$$

$$\text{ec}\ \tfrac{1}{2} A = \frac{1 - \cos A}{(1 + \cos A) + \sqrt{2 (1 + \cos A)}}$$

$$\text{ec}\ 2\, A = \frac{2 \tan^2 A}{1 - \tan^2 A}$$

$$(A \pm B) = \sin A \,.\, \cos B \pm \sin B \,.\, \cos A$$

$$(A \pm B) = \cos A \,.\, \cos B \mp \sin A \,.\, \sin B$$

$$A + \sin B = 2 \sin \tfrac{1}{2} (A + B) \cos \tfrac{1}{2} (A - B)$$

$$A - \sin B = 2 \cos \tfrac{1}{2} (A + B) \sin \tfrac{1}{2} (A - B)$$

$$A + \cos B = 2 \cos \tfrac{1}{2} (A + B) \cos \tfrac{1}{2} (A - B)$$

$$B - \cos A = 2 \sin \tfrac{1}{2} (A + B) \sin \tfrac{1}{2} (A - B)$$

$$A - \sin^2 B = \cos^2 B - \cos^2 A = \sin (A + B) \sin (A - B)$$

$$A - \sin^2 B = \cos (A + B) \cos (A - B)$$

$$A + \tan B = \frac{\sin (A + B)}{\cos A \,.\, \cos B}$$

$$A - \tan B = \frac{\sin (A - B)}{\cos A \,.\, \cos B}$$

′	5°		6°		7°		8°		9°		′
	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	
0	.08716	.99619	.10453	.99452	.12187	.99255	.13917	.99027	.15643	.98769	60
1	.08745	.99617	.10482	.99449	.12216	.99251	.13946	.99023	.15672	.98764	59
2	.08774	.99614	.10511	.99446	.12245	.99248	.13975	.99019	.15701	.98760	58
3	.08803	.99612	.10540	.99443	.12274	.99244	.14004	.99015	.15730	.98755	57
4	.08831	.99609	.10569	.99440	.12302	.99240	.14033	.99011	.15758	.98751	56
5	.08860	.99607	.10597	.99437	.12331	.99237	.14061	.99006	.15787	.98746	55
6	.08889	.99604	.10626	.99434	.12360	.99233	.14090	.99002	.15816	.98741	54
7	.08918	.99602	.10655	.99431	.12389	.99230	.14119	.98998	.15845	.98737	53
8	.08947	.99599	.10684	.99428	.12418	.99226	.14148	.98994	.15873	.98732	52
9	.08976	.99596	.10713	.99424	.12447	.99222	.14177	.98990	.15902	.98728	51
10	.09005	.99594	.10742	.99421	.12476	.99219	.14205	.98986	.15931	.98723	50
11	.09034	.99591	.10771	.99418	.12504	.99215	.14234	.98982	.15959	.98718	49
12	.09063	.99588	.10800	.99415	.12533	.99211	.14263	.98978	.15988	.98714	48
13	.09092	.99586	.10829	.99412	.12562	.99208	.14292	.98973	.16017	.98709	47
14	.09121	.99583	.10858	.99409	.12591	.99204	.14320	.98969	.16046	.98704	46
15	.09150	.99580	.10887	.99406	.12620	.99200	.14349	.98965	.16074	.98700	45
16	.09179	.99578	.10916	.99402	.12649	.99197	.14378	.98961	.16103	.98695	44
17	.09208	.99575	.10945	.99399	.12678	.99193	.14407	.98957	.16132	.98690	43
18	.09237	.99572	.10973	.99396	.12706	.99189	.14436	.98953	.16160	.98686	42
19	.09266	.99570	.11002	.99393	.12735	.99186	.14464	.98948	.16189	.98681	41
20	.09295	.99567	.11031	.99390	.12764	.99182	.14493	.98944	.16218	.98676	40
21	.09324	.99564	.11060	.99386	.12793	.99178	.14522	.98940	.16246	.98671	39
22	.09353	.99562	.11089	.99383	.12822	.99175	.14551	.98936	.16275	.98667	38
23	.09382	.99559	.11118	.99380	.12851	.99171	.14580	.98931	.16304	.98662	37
24	.09411	.99556	.11147	.99377	.12880	.99167	.14608	.98927	.16333	.98657	36
25	.09440	.99553	.11176	.99374	.12908	.99163	.14637	.98923	.16361	.98652	35
26	.09469	.99551	.11205	.99370	.12937	.99160	.14666	.98919	.16390	.98648	34
27	.09498	.99548	.11234	.99367	.12966	.99156	.14695	.98914	.16419	.98643	33
28	.09527	.99545	.11263	.99364	.12995	.99152	.14723	.98910	.16447	.98638	32
29	.09556	.99542	.11291	.99360	.13024	.99148	.14752	.98906	.16476	.98633	31
30	.09585	.99540	.11320	.99357	.13053	.99144	.14781	.98902	.16505	.98629	30
31	.09614	.99537	.11349	.99354	.13081	.99141	.14810	.98897	.16533	.98624	29
32	.09642	.99534	.11378	.99351	.13110	.99137	.14838	.98893	.16562	.98619	28
33	.09671	.99531	.11407	.99347	.13139	.99133	.14867	.98889	.16591	.98614	27
34	.09700	.99528	.11436	.99344	.13168	.99129	.14896	.98884	.16620	.98609	26
35	.09729	.99526	.11465	.99341	.13197	.99125	.14925	.98880	.16648	.98604	25
36	.09758	.99523	.11494	.99337	.13226	.99122	.14954	.98876	.16677	.98600	24
37	.09787	.99520	.11523	.99334	.13254	.99118	.14982	.98871	.16706	.98595	23
38	.09816	.99517	.11552	.99331	.13283	.99114	.15011	.98867	.16734	.98590	22
39	.09845	.99514	.11580	.99327	.13312	.99110	.15040	.98863	.16763	.98585	21
40	.09874	.99511	.11609	.99324	.13341	.99106	.15069	.98858	.16792	.98580	20
41	.09903	.99508	.11638	.99320	.13370	.99102	.15097	.98854	.16820	.98575	19
42	.09932	.99506	.11667	.99317	.13399	.99098	.15126	.98849	.16849	.98570	18
43	.09961	.99503	.11696	.99314	.13427	.99094	.15155	.98845	.16878	.98565	17
44	.09990	.99500	.11725	.99310	.13456	.99091	.15184	.98841	.16906	.98561	16
45	.10019	.99497	.11754	.99307	.13485	.99087	.15212	.98836	.16935	.98556	15
46	.10048	.99494	.11783	.99303	.13514	.99083	.15241	.98832	.16964	.98551	14
47	.10077	.99491	.11812	.99300	.13543	.99079	.15270	.98827	.16992	.98546	13
48	.10106	.99488	.11840	.99297	.13572	.99075	.15299	.98823	.17021	.98541	12
49	.10135	.99485	.11869	.99293	.13600	.99071	.15327	.98818	.17050	.98536	11
50	.10164	.99482	.11898	.99290	.13629	.99067	.15356	.98814	.17078	.98531	10
51	.10192	.99479	.11927	.99286	.13658	.99063	.15385	.98809	.17107	.98526	9
52	.10221	.99476	.11956	.99283	.13687	.99059	.15414	.98805	.17136	.98521	8
53	.10250	.99473	.11985	.99279	.13716	.99055	.15442	.98800	.17164	.98516	7
54	.10279	.99470	.12014	.99276	.13744	.99051	.15471	.98796	.17193	.98511	6
55	.10308	.99467	.12043	.99272	.13773	.99047	.15500	.98791	.17222	.98506	5
56	.10337	.99464	.12071	.99269	.13802	.99043	.15529	.98787	.17250	.98501	4
57	.10366	.99461	.12100	.99265	.13831	.99039	.15557	.98782	.17279	.98496	3
58	.10395	.99458	.12129	.99262	.13860	.99035	.15586	.98778	.17308	.98491	2
59	.10424	.99455	.12158	.99258	.13889	.99031	.15615	.98773	.17336	.98486	1
60	.10453	.99452	.12187	.99255	.13917	.99027	.15643	.98769	.17365	.98481	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	84°		83°		82°		81°		80°		

′	15° Sine	15° Cosin	16° Sine	16° Cosin	17° Sine	17° Cosin	18° Sine	18° Cosin	19° Sine	19° Cosin	′
0	.25882	.96593	.27564	.96126	.29237	.95630	.30902	.95106	.32557	.94552	60
1	.25910	.96585	.27592	.96118	.29265	.95622	.30929	.95097	.32584	.94542	59
2	.25938	.96578	.27620	.96110	.29293	.95613	.30957	.95088	.32612	.94533	58
3	.25966	.96570	.27648	.96102	.29321	.95605	.30985	.95079	.32639	.94523	57
4	.25994	.96562	.27676	.96094	.29348	.95596	.31012	.95070	.32667	.94514	56
5	.26022	.96555	.27704	.96086	.29376	.95588	.31040	.95061	.32694	.94504	55
6	.26050	.96547	.27731	.96078	.29404	.95579	.31068	.95052	.32722	.94495	54
7	.26079	.96540	.27759	.96070	.29432	.95571	.31095	.95043	.32749	.94485	53
8	.26107	.96532	.27787	.96062	.29460	.95562	.31123	.95033	.32777	.94476	52
9	.26135	.96524	.27815	.96054	.29487	.95554	.31151	.95024	.32804	.94466	51
10	.26163	.96517	.27843	.96046	.29515	.95545	.31178	.95015	.32832	.94457	50
11	.26191	.96509	.27871	.96037	.29543	.95536	.31206	.95006	.32859	.94447	49
12	.26219	.96502	.27899	.96029	.29571	.95528	.31233	.94997	.32887	.94438	48
13	.26247	.96494	.27927	.96021	.29599	.95519	.31261	.94988	.32914	.94428	47
14	.26275	.96486	.27955	.96013	.29626	.95511	.31289	.94979	.32942	.94418	46
15	.26303	.96479	.27983	.96005	.29654	.95502	.31316	.94970	.32969	.94409	45
16	.26331	.96471	.28011	.95997	.29682	.95493	.31344	.94961	.32997	.94399	44
17	.26359	.96463	.28039	.95989	.29710	.95485	.31372	.94952	.33024	.94390	43
18	.26387	.96456	.28067	.95981	.29737	.95476	.31399	.94943	.33051	.94380	42
19	.26415	.96448	.28095	.95972	.29765	.95467	.31427	.94933	.33079	.94370	41
20	.26443	.96440	.28123	.95964	.29793	.95459	.31454	.94924	.33106	.94361	40
21	.26471	.96433	.28150	.95956	.29821	.95450	.31482	.94915	.33134	.94351	39
22	.26500	.96425	.28178	.95948	.29849	.95441	.31510	.94906	.33161	.94342	38
23	.26528	.96417	.28206	.95940	.29876	.95433	.31537	.94897	.33189	.94332	37
24	.26556	.96410	.28234	.95931	.29904	.95424	.31565	.94888	.33216	.94322	36
25	.26584	.96402	.28262	.95923	.29932	.95415	.31593	.94878	.33244	.94313	35
26	.26612	.96394	.28290	.95915	.29960	.95407	.31620	.94869	.33271	.94303	34
27	.26640	.96386	.28318	.95907	.29987	.95398	.31648	.94860	.33298	.94293	33
28	.26668	.96379	.28346	.95898	.30015	.95389	.31675	.94851	.33326	.94284	32
29	.26696	.96371	.28374	.95890	.30043	.95380	.31703	.94842	.33353	.94274	31
30	.26724	.96363	.28402	.95882	.30071	.95372	.31730	.94832	.33381	.94264	30
31	.26752	.96355	.28429	.95874	.30098	.95363	.31758	.94823	.33408	.94254	29
32	.26780	.96347	.28457	.95865	.30126	.95354	.31786	.94814	.33436	.94245	28
33	.26808	.96340	.28485	.95857	.30154	.95345	.31813	.94805	.33463	.94235	27
34	.26836	.96332	.28513	.95849	.30182	.95337	.31841	.94795	.33490	.94225	26
35	.26864	.96324	.28541	.95841	.30209	.95328	.31868	.94786	.33518	.94215	25
36	.26892	.96316	.28569	.95832	.30237	.95319	.31896	.94777	.33545	.94206	24
37	.26920	.96308	.28597	.95824	.30265	.95310	.31923	.94768	.33573	.94196	23
38	.26948	.96301	.28625	.95816	.30292	.95301	.31951	.94758	.33600	.94186	22
39	.26976	.96293	.28652	.95807	.30320	.95293	.31979	.94749	.33627	.94176	21
40	.27004	.96285	.28680	.95799	.30348	.95284	.32006	.94740	.33655	.94167	20
41	.27032	.96277	.28708	.95791	.30376	.95275	.32034	.94730	.33682	.94157	19
42	.27060	.96269	.28736	.95782	.30403	.95266	.32061	.94721	.33710	.94147	18
43	.27088	.96261	.28764	.95774	.30431	.95257	.32089	.94712	.33737	.94137	17
44	.27116	.96253	.28792	.95766	.30459	.95248	.32116	.94702	.33764	.94127	16
45	.27144	.96246	.28820	.95757	.30486	.95240	.32144	.94693	.33792	.94118	15
46	.27172	.96238	.28847	.95749	.30514	.95231	.32171	.94684	.33819	.94108	14
47	.27200	.96230	.28875	.95740	.30542	.95222	.32199	.94674	.33846	.94098	13
48	.27228	.96222	.28903	.95732	.30570	.95213	.32227	.94665	.33874	.94088	12
49	.27256	.96214	.28931	.95724	.30597	.95204	.32254	.94656	.33901	.94078	11
50	.27284	.96206	.28959	.95715	.30625	.95195	.32282	.94646	.33929	.94068	10
51	.27312	.96198	.28987	.95707	.30653	.95186	.32309	.94637	.33956	.94058	9
52	.27340	.96190	.29015	.95698	.30680	.95177	.32337	.94627	.33983	.94049	8
53	.27368	.96182	.29042	.95690	.30708	.95168	.32364	.94618	.34011	.94039	7
54	.27396	.96174	.29070	.95681	.30736	.95159	.32392	.94609	.34038	.94029	6
55	.27424	.96166	.29098	.95673	.30763	.95150	.32419	.94599	.34065	.94019	5
56	.27452	.96158	.29126	.95664	.30791	.95142	.32447	.94590	.34093	.94009	4
57	.27480	.96150	.29154	.95656	.30819	.95133	.32474	.94580	.34120	.93999	3
58	.27508	.96142	.29182	.95647	.30846	.95124	.32502	.94571	.34147	.93989	2
59	.27536	.96134	.29209	.95639	.30874	.95115	.32529	.94561	.34175	.93979	1
60	.27564	.96126	.29237	.95630	.30902	.95106	.32557	.94552	.34202	.93969	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	74°		73°		72°		71°		70°		

′	0°		1°	′
	Sine	Cosin	Sine	
0	.00000	One.	.01745	60
1	.00029	One.	.01774	59
2	.00058	One.	.01803	58
3	.00087	One.	.01832	57
4	.00116	One.	.01862	56
5	.00145	One.	.01891	55
6	.00175	One.	.01920	54
7	.00204	One.	.01949	53
8	.00233	One.	.01978	52
9	.00262	One.	.02007	51
10	.00291	One.	.02036	50
11	.00320	.99999	.02065	49
12	.00349	.99999	.02094	48
13	.00378	.99999	.02123	47
14	.00407	.99999	.02152	46
15	.00436	.99999	.02181	45
16	.00465	.99999	.02211	44
17	.00495	.99999	.02240	43
18	.00524	.99999	.02269	42
19	.00553	.99998	.02298	41
20	.00582	.99998	.02327	40
21	.00611	.99998	.02356	39
22	.00640	.99998	.02385	38
23	.00669	.99998	.02414	37
24	.00698	.99998	.02443	36
25	.00727	.99997	.02472	35
26	.00756	.99997	.02501	34
27	.00785	.99997	.02530	33
28	.00814	.99997	.02560	32
29	.00844	.99996	.02589	31
30	.00873	.99996	.02618	30
31	.00902	[redacted]	.02647	29
32	.00931	.99996	.02676	28
33	.00960	.99995	.02705	27
34	.00989	.99995	.02734	26
35	.01018	.99995	.02763	25
36	.01047	.99995	.02792	24
37	.01076	.99994	.02821	23
38	.01105	.99994	.02850	22
39	.01134	.99994	.02879	21
40	.01164	.99993	.02908	20
41	.01193	.99993	.02938	19
42	.01222	.99993	.02967	18
43	.01251	.99992	.02996	17
44	.01280	.99992	.03025	16
45	.01309	.99991	.03054	15
46	.01338	.99991	.03083	14
47	.01367	.99991	.03112	13
48	.01396	.99990	.03141	12
49	.01425	.99990	.03170	11
50	.01454	.99989	.03199	10
51	.01483	.99989	.03228	9
52	.01513	.99989	.03257	8
53	.01542	.99988	.03286	7
54	.01571	.99988	.03316	6
55	.01600	.99987	.03345	5
56	.01629	.99987	.03374	4
57	.01658	.99986	.03403	3
58	.01687	.99986	.03432	2
59	.01716	.99985	.03461	1
60	.01745	.99985	.03490	0
′	Cosin	Sine	Cosin	′
	89°		[redacted]	

5°		6°		7°		8°		9°		′
ne	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	
716	.99619	.10453	.99452	.12187	.99255	.13917	.99027	.15643	.98769	60
745	.99617	.10482	.99449	.12216	.99251	.13946	.99023	.15672	.98764	59
774	.99614	.10511	.99446	.12245	.99248	.13975	.99019	.15701	.98760	58
803	.99612	.10540	.99443	.12274	.99244	.14004	.99015	.15730	.98755	57
831	.99609	.10569	.99440	.12302	.99240	.14033	.99011	.15758	.98751	56
860	.99607	.10597	.99437	.12331	.99237	.14061	.99006	.15787	.98746	55
889	.99604	.10626	.99434	.12360	.99233	.14090	.99002	.15816	.98741	54
918	.99602	.10655	.99431	.12389	.99230	.14119	.98998	.15845	.98737	53
947	.99599	.10684	.99428	.12418	.99226	.14148	.98994	.15873	.98732	52
976	.99596	.10713	.99424	.12447	.99222	.14177	.98990	.15902	.98728	51
005	.99594	.10742	.99421	.12476	.99219	.14205	.98986	.15931	.98723	50
034	.99591	.10771	.99418	.12504	.99215	.14234	.98982	.15959	.98718	49
063	.99588	.10800	.99415	.12533	.99211	.14263	.98978	.15988	.98714	48
092	.99586	.10829	.99412	.12562	.99208	.14292	.98973	.16017	.98709	47
121	.99583	.10858	.99409	.12591	.99204	.14320	.98969	.16046	.98704	46
150	.99580	.10887	.99406	.12620	.99200	.14349	.98965	.16074	.98700	45
179	.99578	.10916	.99402	.12649	.99197	.14378	.98961	.16103	.98695	44
208	.99575	.10945	.99399	.12678	.99193	.14407	.98957	.16132	.98690	43
237	.99572	.10973	.99396	.12706	.99189	.14436	.98953	.16160	.98686	42
266	.99570	.11002	.99393	.12735	.99186	.14464	.98948	.16189	.98681	41
295	.99567	.11031	.99390	.12764	.99182	.14493	.98944	.16218	.98676	40
324	.99564	.11060	.99386	.12793	.99178	.14522	.98940	.16246	.98671	39
353	.99562	.11089	.99383	.12822	.99175	.14551	.98936	.16275	.98667	38
382	.99559	.11118	.99380	.12851	.99171	.14580	.98931	.16304	.98662	37
411	.99556	.11147	.99377	.12880	.99167	.14608	.98927	.16333	.98657	36
440	.99553	.11176	.99374	.12908	.99163	.14637	.98923	.16361	.98652	35
469	.99551	.11205	.99370	.12937	.99160	.14666	.98919	.16390	.98648	34
498	.99548	.11234	.99367	.12966	.99156	.14695	.98914	.16419	.98643	33
527	.99545	.11263	.99364	.12995	.99152	.14723	.98910	.16447	.98638	32
556	.99542	.11291	.99360	.13024	.99148	.14752	.98906	.16476	.98633	31
585	.99540	.11320	.99357	.13053	.99144	.14781	.98902	.16505	.98629	30
614	.99537	.11349	.99354	.13081	.99141	.14810	.98897	.16533	.98624	29
642	.99534	.11378	.99351	.13110	.99137	.14838	.98893	.16562	.98619	28
671	.99531	.11407	.99347	.13139	.99133	.14867	.98889	.16591	.98614	27
700	.99528	.11436	.99344	.13168	.99129	.14896	.98884	.16620	.98609	26
729	.99526	.11465	.99341	.13197	.99125	.14925	.98880	.16648	.98604	25
758	.99523	.11494	.99337	.13226	.99122	.14954	.98876	.16677	.98600	24
787	.99520	.11523	.99334	.13254	.99118	.14982	.98871	.16706	.98595	23
816	.99517	.11552	.99331	.13283	.99114	.15011	.98867	.16734	.98590	22
845	.99514	.11580	.99327	.13312	.99110	.15040	.98863	.16763	.98585	21
874	.99511	.11609	.99324	.13341	.99106	.15069	.98858	.16792	.98580	20
903	.99508	.11638	.99320	.13370	.99102	.15097	.98854	.16820	.98575	19
932	.99506	.11667	.99317	.13399	.99098	.15126	.98849	.16849	.98570	18
961	.99503	.11696	.99314	.13427	.99094	.15155	.98845	.16878	.98565	17
990	.99500	.11725	.99310	.13456	.99091	.15184	.98841	.16906	.98561	16
019	.99497	.11754	.99307	.13485	.99087	.15212	.98836	.16935	.98556	15
048	.99494	.11783	.99303	.13514	.99083	.15241	.98832	.16964	.98551	14
077	.99491	.11812	.99300	.13543	.99079	.15270	.98827	.16992	.98546	13
106	.99488	.11840	.99297	.13572	.99075	.15299	.98823	.17021	.98541	12
135	.99485	.11869	.99293	.13600	.99071	.15327	.98818	.17050	.98536	11
164	.99482	.11898	.99290	.13629	.99067	.15356	.98814	.17078	.98531	10
192	.99479	.11927	.99286	.13658	.99063	.15385	.98809	.17107	.98526	9
221	.99476	.11956	.99283	.13687	.99059	.15414	.98805	.17136	.98521	8
250	.99473	.11985	.99279	.13716	.99055	.15442	.98800	.17164	.98516	7
279	.99470	.12014	.99276	.13744	.99051	.15471	.98796	.17193	.98511	6
308	.99467	.12043	.99272	.13773	.99047	.15500	.98791	.17222	.98506	5
337	.99464	.12071	.99269	.13802	.99043	.15529	.98787	.17250	.98501	4
366	.99461	.12100	.99265	.13831	.99039	.15557	.98782	.17279	.98496	3
395	.99458	.12129	.99262	.13860	.99035	.15586	.98778	.17308	.98491	2
424	.99455	.12158	.99258	.13889	.99031	.15615	.98773	.17336	.98486	1
453	.99452	.12187	.99255	.13917	.99027	.15643	.98769	.17365	.98481	0
sin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
84°		83°		82°		81°		80°		

′	10° Sine	10° Cosin	11° Sine	11° Cosin	12° Sine	12° Cosin	13° Sine	13° Cosin	14° Sine	14° Cosin	′
0	17365	98481	19081	98163		97815	22495	97437		97030	60
1	17393	98476	19109	98157		97809	22523	97430		97023	59
2	17422	98471	19138	98152		97803	22552	97424		97015	58
3	17451	98466	19167	98146		97797	22580	97417		97008	57
4	17479	98461	19195	98140		97791	22608	97411		97001	56
5	17508	98455	19224	98135		97784	22637	97404		96994	55
6	17537	98450	19252	98129		97778	22665	97398		96987	54
7	17565	98445	19281	98124		97772	22693	97391		96980	53
8	17594	98440	19309	98118		97766	22722	97384		96973	52
9	17623	98435	19338	98112		97760	22750	97378		96966	51
10	17651	98430	19366	98107		97754	22778	97371		96959	50
11	17680	98425	19395	98101		97748	22807	97365		96952	49
12	17708	98420	19423	98096		97742	22835	97358		96945	48
13	17737	98414	19452	98090		97735	22863	97351		96937	47
14	17766	98409	19481	98084		97729	22892	97345		96930	46
15	17794	98404	19509	98079		97723	22920	97338		96923	45
16	17823	98399	19538	98073		97717	22948	97331		96916	44
17	17852	98394	19566	98067		97711	22977	97325		96909	43
18	17880	98389	19595	98061		97705	23005	97318		96902	42
19	17909	98383	19623	98056		97698	23033	97311		96894	41
20	17937	98378	19652	98050		97692	23062	97304		96887	40
21	17966	98373	19680	98044		97686	23090	97298		96880	39
22	17995	98368	19709	98039		97680	23118	97291		96873	38
23	18023	98362	19737	98033		97673	23146	97284		96866	37
24	18052	98357	19766	98027		97667	23175	97278		96858	36
25	18081	98352	19794	98021		97661	23203	97271		96851	35
26	18109	98347	19823	98016		97655	23231	97264		96844	34
27	18138	98341	19851	98010		97648	23260	97257		96837	33
28	18166	98336	19880	98004		97642	23288	97251		96829	32
29	18195	98331	19908	97998		97636	23316	97244		96822	31
30	18224	98325	19937	97992		97630	23345	97237		96815	30
31	18252	98320	19965	97987		97623	23373	97230		96807	29
32	18281	98315	19994	97981		97617	23401	97223		96800	28
33	18309	98310	20022	97975		97611	23429	97217		96793	27
34	18338	98304	20051	97969		97604	23458	97210		96786	26
35	18367	98299	20079	97963		97598	23486	97203		96778	25
36	18395	98294	20108	97958		97592	23514	97196		96771	24
37	18424	98288	20136	97952		97585	23542	97189		96764	23
38	18452	98283	20165	97946		97579	23571	97182		96756	22
39	18481	98277	20193	97940		97573	23599	97176		96749	21
40	18509	98272	20222	97934		97566	23627	97169	25320	96742	20
41	18538	98267	20250	97928		97560	23656	97162	25348	96734	19
42	18567	98261	20279	97922		97553	23684	97155	25376	96727	18
43	18595	98256	20307	97916		97547	23712	97148	25404	96719	17
44	18624	98250	20336	97910		97541	23740	97141	25432	96712	16
45	18652	98245	20364	97905		97534	23769	97134	25460	96705	15
46	18681	98240	20393	97899		97528	23797	97127	25488	96697	14
47	18710	98234	20421	97893	[illegible]	97521	23825	97120	25516	96690	13
48	18738	98229	20450	97887	[illegible]	97515	23853	97113	25545	96682	12
49	18767	98223	20478	97881	[illegible]	97508	23882	97106	25573	96675	11
50	18795	98218	20507	97875	[illegible]	97502	23910	97100	25601	96667	10
51	18824	98212	20535	97869	22240	97496	23938	97093	25629	96660	9
52	18852	98207	20563	97863	22268	97489	23966	97086	25657	96653	8
53	18881	98201	20592	97857	22297	97483	23995	97079	25685	96645	7
54	18910	98196	20620	97851	22325	97476	24023	97072	25713	96638	6
55	18938	98190	20649	97845	22353	97470	24051	97065	25741	96630	5
56	18967	98185	20677	97839	22382	97463	24079	97058	25769	96623	4
57	18995	98179	20706	97833	22410	97457	24108	97051	25798	96615	3
58	19024	98174	20734	97827	22438	97450	24136	97044	25826	96608	2
59	19052	98168	20763	97821	22467	97444	24164	97037	25854	96600	1
60	19081	98163	20791	97815	22495	97437	24192	97030	25882	96593	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	79°		78°		77°		76°		75°		

312	.96199	.28987	.95707	.30658	.95186	.32309	.94637	9
340	.96190	.29015	.95698	.30680	.95177	.32337	.94627	8
368	.96182	.29042	.95690	.30708	.95168	.32364	.94618	7
396	.96174	.29070	.95681	.30736	.95159	.32392	.94609	6
424	.96166	.29098	.95673	.30763	.95150	.32419	.94599	5
452	.96158	.29126	.95664	.30791	.95142	.32447	.94590	4
480	.96150	.29154	.95656	.30819	.95133	.32474	.94580	3
508	.96142	.29182	.95647	.30846	.95124	.32502	.94571	2
536	.96134	.29209	.95639	.30874	.95115	.32529	.94561	1
564	.96126	.29237	.95630	.30902	.95106	.32557	.94552	0
sin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
74°		73°		72°		71°		

′	30° Sine	30° Cosin	31° Sine	31° Cosin	32° Sine	32° Cosin	33° Sine	33° Cosin	34° Sine	34° Cosin	′
0	.50000	.86603	.51504	.85717	.52992	.84805	.54464	.83867	.55919	.82904	60
1	.50025	.86588	.51529	.85702	.53017	.84789	.54488	.83851	.55943	.82887	59
2	.50050	.86573	.51554	.85687	.53041	.84774	.54513	.83835	.55968	.82871	58
3	.50076	.86559	.51579	.85672	.53066	.84759	.54537	.83819	.55992	.82855	57
4	.50101	.86544	.51604	.85657	.53091	.84743	.54561	.83804	.56016	.82839	56
5	.50126	.86530	.51628	.85642	.53115	.84728	.54586	.83788	.56040	.82822	55
6	.50151	.86515	.51653	.85627	.53140	.84712	.54610	.83772	.56064	.82806	54
7	.50176	.86501	.51678	.85612	.53164	.84697	.54635	.83756	.56088	.82790	53
8	.50201	.86486	.51703	.85597	.53189	.84681	.54659	.83740	.56112	.82773	52
9	.50227	.86471	.51728	.85582	.53214	.84666	.54683	.83724	.56136	.82757	51
10	.50252	.86457	.51753	.85567	.53238	.84650	.54708	.83708	.56160	.82741	50
11	.50277	.86442	.51778	.85551	.53263	.84635	.54732	.83692	.56184	.82724	49
12	.50302	.86427	.51803	.85536	.53288	.84619	.54756	.83676	.56208	.82708	48
13	.50327	.86413	.51828	.85521	.53312	.84604	.54781	.83660	.56232	.82692	47
14	.50352	.86398	.51852	.85506	.53337	.84588	.54805	.83645	.56256	.82675	46
15	.50377	.86384	.51877	.85491	.53361	.84573	.54829	.83629	.56280	.82659	45
16	.50403	.86369	.51902	.85476	.53386	.84557	.54854	.83613	.56305	.82643	44
17	.50428	.86354	.51927	.85461	.53411	.84542	.54878	.83597	.56329	.82626	43
18	.50453	.86340	.51952	.85446	.53435	.84526	.54902	.83581	.56353	.82610	42
19	.50478	.86325	.51977	.85431	.53460	.84511	.54927	.83565	.56377	.82593	41
20	.50503	.86310	.52002	.85416	.53484	.84495	.54951	.83549	.56401	.82577	40
21	.50528	.86295	.52026	.85401	.53509	.84480	.54975	.83533	.56425	.82561	39
22	.50553	.86281	.52051	.85385	.53534	.84464	.54999	.83517	.56449	.82544	38
23	.50578	.86266	.52076	.85370	.53558	.84448	.55024	.83501	.56473	.82528	37
24	.50603	.86251	.52101	.85355	.53583	.84433	.55048	.83485	.56497	.82511	36
25	.50628	.86237	.52126	.85340	.53607	.84417	.55072	.83469	.56521	.82495	35
26	.50654	.86222	.52151	.85325	.53632	.84402	.55097	.83453	.56545	.82478	34
27	.50679	.86207	.52175	.85310	.53656	.84386	.55121	.83437	.56569	.82462	33
28	.50704	.86192	.52200	.85294	.53681	.84370	.55145	.83421	.56593	.82446	32
29	.50729	.86178	.52225	.85279	.53705	.84355	.55169	.83405	.56617	.82429	31
30	.50754	.86163	.52250	.85264	.53730	.84339	.55194	.83389	.56641	.82413	30
31	.50779	.86148	.52275	.85249	.53754	.84324	.55218	.83373	.56665	.82396	29
32	.50804	.86133	.52299	.85234	.53779	.84308	.55242	.83356	.56689	.82380	28
33	.50829	.86119	.52324	.85218	.53804	.84292	.55266	.83340	.56713	.82363	27
34	.50854	.86104	.52349	.85203	.53828	.84277	.55291	.83324	.56736	.82347	26
35	.50879	.86089	.52374	.85188	.53853	.84261	.55315	.83308	.56760	.82330	25
36	.50904	.86074	.52399	.85173	.53877	.84245	.55339	.83292	.56784	.82314	24
37	.50929	.86059	.52423	.85157	.53902	.84230	.55363	.83276	.56808	.82297	23
38	.50954	.86045	.52448	.85142	.53926	.84214	.55388	.83260	.56832	.82281	22
39	.50979	.86030	.52473	.85127	.53951	.84198	.55412	.83244	.56856	.82264	21
40	.51004	.86015	.52498	.85112	.53975	.84182	.55436	.83228	.56880	.82248	20
41	.51029	.86000	.52522	.85096	.54000	.84167	.55460	.83212	.56904	.82231	19
42	.51054	.85985	.52547	.85081	.54024	.84151	.55484	.83195	.56928	.82214	18
43	.51079	.85970	.52572	.85066	.54049	.84135	.55509	.83179	.56952	.82198	17
44	.51104	.85956	.52597	.85051	.54073	.84120	.55533	.83163	.56976	.82181	16
45	.51129	.85941	.52621	.85035	.54097	.84104	.55557	.83147	.57000	.82165	15
46	.51154	.85926	.52646	.85020	.54122	.84088	.55581	.83131	.57024	.82148	14
47	.51179	.85911	.52671	.85005	.54146	.84072	.55605	.83115	.57047	.82132	13
48	.51204	.85896	.52696	.84989	.54171	.84057	.55630	.83098	.57071	.82115	12
49	.51229	.85881	.52720	.84974	.54195	.84041	.55654	.83082	.57095	.82098	11
50	.51254	.85866	.52745	.84959	.54220	.84025	.55678	.83066	.57119	.82082	10
51	.51279	.85851	.52770	.84943	.54244	.84009	.55702	.83050	.57143	.82065	9
52	.51304	.85836	.52794	.84928	.54269	.83994	.55726	.83034	.57167	.82048	8
53	.51329	.85821	.52819	.84913	.54293	.83978	.55750	.83017	.57191	.82032	7
54	.51354	.85806	.52844	.84897	.54317	.83962	.55775	.83001	.57215	.82015	6
55	.51379	.85792	.52869	.84882	.54342	.83946	.55799	.82985	.57238	.81999	5
56	.51404	.85777	.52893	.84866	.54366	.83930	.55823	.82969	.57262	.81982	4
57	.51429	.85762	.52918	.84851	.54391	.83915	.55847	.82953	.57286	.81965	3
58	.51454	.85747	.52943	.84836	.54415	.83899	.55871	.82936	.57310	.81949	2
59	.51479	.85732	.52967	.84820	.54440	.83883	.55895	.82920	.57334	.81932	1
60	.51504	.85717	.52992	.84805	.54464	.83867	.55919	.82904	.57358	.81915	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	59°		58°		57°		56°		55°		

′	35°		36°		37°		38°		39°		′
	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	
0	.57358	.81915	.58779	.80902	.60182	.79864	.61566	.78801	.62932	.77715	60
1	.57381	.81899	.58802	.80885	.60205	.79846	.61589	.78783	.62955	.77696	59
2	.57405	.81882	.58826	.80867	.60228	.79829	.61612	.78765	.62977	.77678	58
3	.57429	.81865	.58849	.80850	.60251	.79811	.61635	.78747	.63000	.77660	57
4	.57453	.81848	.58873	.80833	.60274	.79793	.61658	.78729	.63022	.77641	56
5	.57477	.81832	.58896	.80816	.60298	.79776	.61681	.78711	.63045	.77623	55
6	.57501	.81815	.58920	.80799	.60321	.79758	.61704	.78694	.63068	.77605	54
7	.57524	.81798	.58943	.80782	.60344	.79741	.61726	.78676	.63090	.77586	53
8	.57548	.81782	.58967	.80765	.60367	.79723	.61749	.78658	.63113	.77568	52
9	.57572	.81765	.58990	.80748	.60390	.79706	.61772	.78640	.63135	.77550	51
10	.57596	.81748	.59014	.80730	.60414	.79688	.61795	.78622	.63158	.77531	50
11	.57619	.81731	.59037	.80713	.60437	.79671	.61818	.78604	.63180	.77513	49
12	.57643	.81714	.59061	.80696	.60460	.79653	.61841	.78586	.63203	.77494	48
13	.57667	.81698	.59084	.80679	.60483	.79635	.61864	.78568	.63225	.77476	47
14	.57691	.81681	.59108	.80662	.60506	.79618	.61887	.78550	.63248	.77458	46
15	.57715	.81664	.59131	.80644	.60529	.79600	.61909	.78532	.63271	.77439	45
16	.57738	.81647	.59154	.80627	.60553	.79583	.61932	.78514	.63293	.77421	44
17	.57762	.81631	.59178	.80610	.60576	.79565	.61955	.78496	.63316	.77402	43
18	.57786	.81614	.59201	.80593	.60599	.79547	.61978	.78478	.63338	.77384	42
19	.57810	.81597	.59225	.80576	.60622	.79530	.62001	.78460	.63361	.77366	41
20	.57833	.81580	.59248	.80558	.60645	.79512	.62024	.78442	.63383	.77347	40
21	.57857	.81563	.59272	.80541	.60668	.79494	.62046	.78424	.63406	.77329	39
22	.57881	.81546	.59295	.80524	.60691	.79477	.62069	.78405	.63428	.77310	38
23	.57904	.81530	.59318	.80507	.60714	.79459	.62092	.78387	.63451	.77292	37
24	.57928	.81513	.59342	.80489	.60738	.79441	.62115	.78369	.63473	.77273	36
25	.57952	.81496	.59365	.80472	.60761	.79424	.62138	.78351	.63496	.77255	35
26	.57976	.81479	.59389	.80455	.60784	.79406	.62160	.78333	.63518	.77236	34
27	.57999	.81462	.59412	.80438	.60807	.79388	.62183	.78315	.63540	.77218	33
28	.58023	.81445	.59436	.80420	.60830	.79371	.62206	.78297	.63563	.77199	32
29	.58047	.81428	.59459	.80403	.60853	.79353	.62229	.78279	.63585	.77181	31
30	.58070	.81412	.59482	.80386	.60876	.79335	.62251	.78261	.63608	.77162	30
31	.58094	.81395	.59506	.80368	.60899	.79318	.62274	.78243	.63630	.77144	29
32	.58118	.81378	.59529	.80351	.60922	.79300	.62297	.78225	.63653	.77125	28
33	.58141	.81361	.59552	.80334	.60945	.79282	.62320	.78206	.63675	.77107	27
34	.58165	.81344	.59576	.80316	.60968	.79264	.62342	.78188	.63698	.77088	26
35	.58189	.81327	.59599	.80299	.60991	.79247	.62365	.78170	.63720	.77070	25
36	.58212	.81310	.59622	.80282	.61015	.79229	.62388	.78152	.63742	.77051	24
37	.58236	.81293	.59646	.80264	.61038	.79211	.62411	.78134	.63765	.77033	23
38	.58260	.81276	.59669	.80247	.61061	.79193	.62433	.78116	.63787	.77014	22
39	.58283	.81259	.59693	.80230	.61084	.79176	.62456	.78098	.63810	.76996	21
40	.58307	.81242	.59716	.80212	.61107	.79158	.62479	.78079	.63832	.76977	20
41	.58330	.81225	.59739	.80195	.61130	.79140	.62502	.78061	.63854	.76959	19
42	.58354	.81208	.59763	.80178	.61153	.79122	.62524	.78043	.63877	.76940	18
43	.58378	.81191	.59786	.80160	.61176	.79105	.62547	.78025	.63899	.76921	17
44	.58401	.81174	.59809	.80143	.61199	.79087	.62570	.78007	.63922	.76903	16
45	.58425	.81157	.59832	.80125	.61222	.79069	.62592	.77988	.63944	.76884	15
46	.58449	.81140	.59856	.80108	.61245	.79051	.62615	.77970	.63966	.76866	14
47	.58472	.81123	.59879	.80091	.61268	.79033	.62638	.77952	.63989	.76847	13
48	.58496	.81106	.59902	.80073	.61291	.79016	.62660	.77934	.64011	.76828	12
49	.58519	.81089	.59926	.80056	.61314	.78998	.62683	.77916	.64033	.76810	11
50	.58543	.81072	.59949	.80038	.61337	.78980	.62706	.77897	.64056	.76791	10
51	.58567	.81055	.59972	.80021	.61360	.78962	.62728	.77879	.64078	.76772	9
52	.58590	.81038	.59995	.80003	.61383	.78944	.62751	.77861	.64100	.76754	8
53	.58614	.81021	.60019	.79986	.61406	.78926	.62774	.77843	.64123	.76735	7
54	.58637	.81004	.60042	.79968	.61429	.78908	.62796	.77824	.64145	.76717	6
55	.58661	.80987	.60065	.79951	.61451	.78891	.62819	.77806	.64167	.76698	5
56	.58684	.80970	.60089	.79934	.61474	.78873	.62842	.77788	.64190	.76679	4
57	.58708	.80953	.60112	.79916	.61497	.78855	.62864	.77769	.64212	.76661	3
58	.58731	.80936	.60135	.79899	.61520	.78837	.62887	.77751	.64234	.76642	2
59	.58755	.80919	.60158	.79881	.61543	.78819	.62909	.77733	.64256	.76623	1
60	.58779	.80902	.60182	.79864	.61566	.78801	.62932	.77715	.64279	.76604	0
′	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	Cosin	Sine	′
	54°		53°		52°		51°		50°		

′	4°		5°		6°		7°	
	Tang		Tang	Cotang	Tang	Cotang	Tang	Co
0	.06993		.08749	11.4301	.10510	9.51436	.12278	8.
1	.07022		.08778	11.3919	.10540	9.48781	.12308	8.
2	.07051		.08807	11.3540	.10569	9.46141	.12338	8.
3	.07080		.08837	11.3163	.10599	9.43515	.12367	8.
4	.07110		.08866	11.2789	.10628	9.40904	.12397	8.
5	.07139		.08895	11.2417	.10657	9.38307	.12426	8.
6	.07168		.08925	11.2048	.10687	9.35724	.12456	8.
7	.07197		.08954	11.1681	.10716	9.33155	.12485	8.
8	.07227		.08983	11.1316	.10746	9.30599	.12515	7.
9	.07256		.09013	11.0954	.10775	9.28058	.12544	7.
10	.07285		.09042	11.0594	.10805	9.25530	.12574	7.
11	.07314		.09071	11.0237	.10834	9.23016	.12603	7.
12	.07344		.09101	10.9882	.10863	9.20516	.12633	7.
13	.07373		.09130	10.9529	.10893	[illegible]	.12662	7.
14	.07402		.09159	10.9178	.10922	9.15554	.12692	7.
15	.07431		.09189	10.8829	.10952	9.13093	.12722	7.
16	.07461		.09218	10.8483	.10981	9.10646	.12751	7.
17	.07490		.09247	10.8139	.11011	9.08211	.12781	7.
18	.07519		.09277	10.7797	.11040	9.05789	.12810	7.
19	.07548		.09306	10.7457	.11070	9.03379	.12840	7.
20	.07578		.09335	10.7119	.11099	9.00983	[illegible]	7.
21	.07607		.09365	10.6783	.11128	8.98598	[illegible]	7.
22	.07636		.09394	10.6450	.11158	8.96227	.12929	7.
23	.07665		.09423	10.6118	.11187	8.93867	.12958	7.
24	.07695		.09453	10.5789	.11217	8.91520	.12988	7.
25	.07724		.09482	10.5462	.11246	8.89185	.13017	7.
26	.07753		.09511	10.5136	.11276	8.86862	.13047	7.
27	.07782		.09541	10.4813	.11305	8.84551	.13076	7.
28	.07812		.09570	10.4491	.11335	8.82252	.13106	7.
29	.07841		.09600	10.4172	.11364	8.79964	.13136	7.
30	.07870		.09629	10.3854	.11394	8.77689	.13165	7.
31	.07899		.09658	10.3538	.11423	8.75425	.13195	7.
32	.07929		.09688	10.3224	.11452	8.73172	.13224	7.
33	.07958		.09717	10.2913	.11482	8.70931	.13254	7.
34	.07987		.09746	10.2602	.11511	8.68701	.13284	7.
35	.08017		.09776	10.2294	.11541	8.66482	.13313	7.
36	.08046		.09805	10.1988	.11570	8.64275	.13343	7.
37	.08075		.09834	10.1683	.11600	8.62078	.13372	7.
38	.08104		.09864	10.1381	.11629	8.59893	.13402	7.
39	.08134		.09893	10.1080	.11659	8.57718	.13432	7.
40	.08163		.09923	10.0780	.11688	8.55555	.13461	7.
41	.08192		.09952	10.0483	.11718	8.53402	.13491	7.
42	.08221		.09981	10.0187	.11747	8.51259	.13521	7.
43	.08251		.10011	9.98931	.11777	8.49128	.13550	7.
44	.08280		.10040	9.96007	.11806	8.47007	.13580	7.
45	.08309		.10069	9.93101	.11836	8.44896	.13609	7.
46	.08339		.10099	9.90211	.11865	8.42795	.13639	7.
47	.08368		.10128	9.87338	.11895	8.40705	.13669	7.
48	.08397		.10158	9.84482	.11924	8.38625	.13698	7.
49	.08427		.10187	9.81641	.11954	8.36555	.13728	7.
50	.08456		.10216	9.78817	.11983	8.34496	.13758	7.
51	.08485		.10246	9.76009	.12013	8.32446	.13787	7.
52	.08514		.10275	9.73217	.12042	8.30406	.13817	7.
53	.08544		.10305	9.70441	.12072	8.28376	.13846	7.
54	.08573		.10334	9.67680	.12101	8.26355	.13876	7.
55	.08602		.10363	9.64935	.12131	8.24345	.13906	7.
56	.08632		.10393	9.62205	.12160	8.22344	.13935	7.
57	.08661		.10422	9.59490	.12190	8.20352	.13965	7.
58	.08690		.10452	9.56791	.12219	8.18370	.13995	7.
59	.08720		.10481	9.54106	.12249	[illegible]	.14024	7.
60	.08749		.10510	9.51436	.12278	8.14435	[illegible]	7.
′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	T
	85°		84°		83°		82°	

8°		9°		10°		11°		′
Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	
14054	7.11537	.15838	6.31375	.17633	5.67128	.19438	5.14455	60
14084	7.10038	.15868	6.30189	.17663	5.66165	.19468	5.13658	59
4113	7.08546	.15898	6.29007	.17693	5.65205	.19498	5.12862	58
4143	7.07059	.15928	6.27829	.17723	5.64248	.19529	5.12069	57
4173	7.05579	.15958	6.26655	.17753	5.63295	.19559	5.11279	56
4202	7.04105	.15988	6.25486	.17783	5.62344	.19589	5.10490	55
4232	7.02637	.16017	6.24321	.17813	5.61397	.19619	5.09704	54
4262	7.01174	.16047	6.23160	.17843	5.60452	.19649	5.08921	53
4291	6.99718	.16077	6.22003	.17873	5.59511	.19680	5.08139	52
4321	6.98268	.16107	6.20851	.17903	5.58573	.19710	5.07360	51
4351	6.96823	.16137	6.19703	.17933	5.57638	.19740	5.06584	50
4381	6.95385	.16167	6.18559	.17963	5.56706	.19770	5.05809	49
4410	6.93952	.16196	6.17419	.17993	5.55777	.19801	5.05037	48
4440	6.92525	.16226	6.16283	.18023	5.54851	.19831	5.04267	47
4470	6.91104	.16256	6.15151	.18053	5.53927	.19861	5.03499	46
4499	6.89688	.16286	6.14023	.18083	5.53007	.19891	5.02734	45
4529	6.88278	.16316	6.12899	.18113	5.52090	.19921	5.01971	44
4559	6.86874	.16346	6.11779	.18143	5.51176	.19952	5.01210	43
4588	6.85475	.16376	6.10664	.18173	5.50264	.19982	5.00451	42
4618	6.84082	.16405	6.09552	.18203	5.49356	.20012	4.99695	41
4648	6.82694	.16435	6.08444	.18233	5.48451	.20042	4.98940	40
4678	6.81312	.16465	6.07340	.18263	5.47548	.20073	4.98188	39
4707	6.79936	.16495	6.06240	.18293	5.46648	.20103	4.97438	38
4737	6.78564	.16525	6.05143	.18323	5.45751	.20133	4.96690	37
4767	6.77199	.16555	6.04051	.18353	5.44857	.20164	4.95945	36
4796	6.75838	.16585	6.02962	.18384	5.43966	.20194	4.95201	35
4826	6.74483	.16615	6.01878	.18414	5.43077	.20224	4.94460	34
4856	6.73133	.16645	6.00797	.18444	5.42192	.20254	4.93721	33
4886	6.71789	.16674	5.99720	.18474	5.41309	.20285	4.92984	32
4915	6.70450	.16704	5.98646	.18504	5.40429	.20315	4.92249	31
4945	6.69116	.16734	5.97576	.18534	5.39552	.20345	4.91516	30
4975	6.67787	.16764	5.96510	.18564	5.38677	.20376	4.90785	29
5005	6.66463	.16794	5.95448	.18594	5.37805	.20406	4.90056	28
5034	6.65144	.16824	5.94390	.18624	5.36936	.20436	4.89330	27
5064	6.63831	.16854	5.93335	.18654	5.36070	.20466	4.88605	26
5094	6.62523	.16884	5.92283	.18684	5.35206	.20497	4.87882	25
5124	6.61219	.16914	5.91236	.18714	5.34345	.20527	4.87162	24
5153	6.59921	.16944	5.90191	.18745	5.33487	.20557	4.86444	23
5183	6.58627	.16974	5.89151	.18775	5.32631	.20588	4.85727	22
5213	6.57339	.17004	5.88114	.18805	5.31778	.20618	4.85013	21
5243	6.56055	.17033	5.87080	.18835	5.30928	.20648	4.84300	20
5272	6.54777	.17063	5.86051	.18865	5.30080	.20679	4.83590	19
5302	6.53503	.17093	5.85024	.18895	5.29235	.20709	4.82882	18
5332	6.52234	.17123	5.84001	.18925	5.28393	.20739	4.82175	17
5362	6.50970	.17153	5.82982	.18955	5.27553	.20770	4.81471	16
5391	6.49710	.17183	5.81966	.18986	5.26715	.20800	4.80769	15
5421	6.48456	.17213	5.80953	.19016	5.25880	.20830	4.80068	14
5451	6.47206	.17243	5.79944	.19046	5.25048	.20861	4.79370	13
5481	6.45961	.17273	5.78938	.19076	5.24218	.20891	4.78673	12
5511	6.44720	.17303	5.77936	.19106	5.23391	.20921	4.77978	11
5540	6.43484	.17333	5.76937	.19136	5.22566	.20952	4.77286	10
5570	6.42253	.17363	5.75941	.19166	5.21744	.20982	4.76595	9
5600	6.41026	.17393	5.74949	.19197	5.20925	.21013	4.75906	8
5630	6.39804	.17423	5.73960	.19227	5.20107	.21043	4.75219	7
5660	6.38587	.17453	5.72974	.19257	5.19293	.21073	4.74534	6
5689	6.37374	.17483	5.71992	.19287	5.18480	.21104	4.73851	5
5719	6.36165	.17513	5.71013	.19317	5.17671	.21134	4.73170	4
5749	6.34961	.17543	5.70037	.19347	5.16863	.21164	4.72490	3
5779	6.33761	.17573	5.69064	.19378	5.16058	.21195	4.71813	2
5809	6.32566	.17603	5.68094	.19408	5.15256	.21225	4.71137	1
5838	6.31375	.17633	5.67128	.19438	5.14455	.21256	4.70463	0
Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	′
81°		80°		79°		78°		

′	12° Tang	12° Cotang	
0	.21256	4.70463	60
1	.21286	4.69791	59
2	.21316	4.69121	58
3	.21347	4.68452	57
4	.21377	4.67786	56
5	.21408	4.67121	55
6	.21438	4.66458	54
7	.21469	4.65797	53
8	.21499	4.65138	52
9	.21529	4.64480	51
10	.21560	4.63825	50
11	.21590	4.63171	49
12	.21621	4.62518	48
13	.21651	4.61868	47
14	.21682	4.61219	46
15	.21712	4.60572	45
16	.21743	4.59927	44
17	.21773	4.59283	43
18	.21804	4.58641	42
19	.21834	4.58001	41
20	.21864	4.57363	40
21	.21895	4.56726	39
22	.21925	4.56091	38
23	.21956	4.55458	37
24	.21986	4.54826	36
25	.22017	4.54196	35
26	.22047	4.53568	34
27	.22078	4.52941	33
28	.22108	4.52316	32
29	.22139	4.51693	31
30	.22169	4.51071	30
31	.22200	4.50451	29
32	.22231	4.49832	28
33	.22261	4.49215	27
34	.22292	4.48600	26
35	.22322	4.47986	25
36	.22353	4.47374	24
37	.22383	4.46764	23
38	.22414	4.46155	22
39	.22444	4.45548	21
40	.22475	4.44942	20
41	.22505	4.44338	19
42	.22536	4.43735	18
43	.22567	4.43134	17
44	.22597	4.42534	16
45	.22628	4.41936	15
46	.22658	4.41340	14
47	.22689	4.40745	13
48	.22719	4.40152	12
49	.22750	4.39560	11
50	.22781	4.38969	10
51	.22811	4.38381	9
52	.22842	4.37793	8
53	.22872	4.37207	7
54	.22903	4.36623	6
55	.22934	4.36040	5
56	.22964	4.35459	4
57	.22995	4.34879	3
58	.23026	4.34300	2
59	.23056	4.33723	1
60	.23087	4.33148	0
′	77° Cotang	77° Tang	′

16°		17°		18°		19°		′
ng	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	
75	3.48741	.30573	3.27085	.32492	3.07768	.34433	2.90421	60
06	3.48359	.30605	3.26745	.32524	3.07464	.34465	2.90147	59
38	3.47977	.30637	3.26406	.32556	3.07160	.34498	2.89873	58
69	3.47596	.30669	3.26067	.32588	3.06857	.34530	2.89600	57
00	3.47216	.30700	3.25729	.32621	3.06554	.34563	2.89327	56
32	3.46837	.30732	3.25392	.32653	3.06252	.34596	2.89055	55
64	3.46458	.30764	3.25055	.32685	3.05950	.34628	2.88783	54
95	3.46080	.30796	3.24719	.32717	3.05649	.34661	2.88511	53
27	3.45703	.30828	3.24383	.32749	3.05349	.34693	2.88240	52
58	3.45327	.30860	3.24049	.32782	3.05049	.34726	2.87970	51
90	3.44951	.30891	3.23714	.32814	3.04749	.34758	2.87700	50
21	3.44576	.30923	3.23381	.32846	3.04450	.34791	2.87430	49
53	3.44202	.30955	3.23048	.32878	3.04152	.34824	2.87161	48
84	3.43829	.30987	3.22715	.32911	3.03854	.34856	2.86892	47
16	3.43456	.31019	3.22384	.32943	3.03556	.34889	2.86624	46
47	3.43084	.31051	3.22053	.32975	3.03260	.34922	2.86356	45
79	3.42713	.31083	3.21722	.33007	3.02963	.34954	2.86089	44
10	3.42343	.31115	3.21392	.33040	3.02667	.34987	2.85822	43
42	3.41973	.31147	3.21063	.33072	3.02372	.35020	2.85555	42
74	3.41604	.31178	3.20734	.33104	3.02077	.35052	2.85289	41
05	3.41236	.31210	3.20406	.33136	3.01783	.35085	2.85023	40
37	3.40869	.31242	3.20079	.33169	3.01489	.35118	2.84758	39
68	3.40502	.31274	3.19752	.33201	3.01196	.35150	2.84494	38
00	3.40136	.31306	3.19426	.33233	3.00903	.35183	2.84229	37
32	3.39771	.31338	3.19100	.33266	3.00611	.35216	2.83965	36
63	3.39406	.31370	3.18775	.33298	3.00319	.35248	2.83702	35
95	3.39042	.31402	3.18451	.33330	3.00028	.35281	2.83439	34
26	3.38679	.31434	3.18127	.33363	2.99738	.35314	2.83176	33
58	3.38317	.31466	3.17804	.33395	2.99447	.35346	2.82914	32
90	3.37955	.31498	3.17481	.33427	2.99158	.35379	2.82653	31
21	3.37594	.31530	3.17159	.33460	2.98868	.35412	2.82391	30
53	3.37234	.31562	3.16838	.33492	2.98580	.35445	2.82130	29
85	3.36875	.31594	3.16517	.33524	2.98292	.35477	2.81870	28
16	3.36516	.31626	3.16197	.33557	2.98004	.35510	2.81610	27
48	3.36158	.31658	3.15877	.33589	2.97717	.35543	2.81350	26
80	3.35800	.31690	3.15558	.33621	2.97430	.35576	2.81091	25
11	3.35443	.31722	3.15240	.33654	2.97144	.35608	2.80833	24
43	3.35087	.31754	3.14922	.33686	2.96858	.35641	2.80574	23
75	3.34732	.31786	3.14605	.33718	2.96573	.35674	2.80316	22
06	3.34377	.31818	3.14288	.33751	2.96288	.35707	2.80059	21
38	3.34023	.31850	3.13972	.33783	2.96004	.35740	2.79802	20
70	3.33670	.31882	3.13656	.33816	2.95721	.35772	2.79545	19
01	3.33317	.31914	3.13341	.33848	2.95437	.35805	2.79289	18
33	3.32965	.31946	3.13027	.33881	2.95155	.35838	2.79033	17
65	3.32614	.31978	3.12713	.33913	2.94872	.35871	2.78778	16
97	3.32264	.32010	3.12400	.33945	2.94591	.35904	2.78523	15
28	3.31914	.32042	3.12087	.33978	2.94309	.35937	2.78269	14
60	3.31565	.32074	3.11775	.34010	2.94028	.35969	2.78014	13
92	3.31216	.32106	3.11464	.34043	2.93748	.36002	2.77761	12
24	3.30868	.32139	3.11153	.34075	2.93468	.36035	2.77507	11
55	3.30521	.32171	3.10842	.34108	2.93189	.36068	2.77254	10
87	3.30174	.32203	3.10532	.34140	2.92910	.36101	2.77002	9
19	3.29829	.32235	3.10223	.34173	2.92632	.36134	2.76750	8
51	3.29483	.32267	3.09914	.34205	2.92354	.36167	2.76498	7
82	3.29139	.32299	3.09606	.34238	2.92076	.36199	2.76247	6
14	3.28795	.32331	3.09298	.34270	2.91799	.36232	2.75996	5
46	3.28452	.32363	3.08991	.34303	2.91523	.36265	2.75746	4
78	3.28109	.32396	3.08685	.34335	2.91246	.36298	2.75496	3
09	3.27767	.32428	3.08379	.34368	2.90971	.36331	2.75246	2
41	3.27426	.32460	3.08073	.34400	2.90696	.36364	2.74997	1
73	3.27085	.32492	3.07768	.34433	2.90421	.36397	2.74748	0
ng	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	′
73°		72°		71°		70°		

′	20° Tang	20° Cotang	21° Tang	21° Cotang	22° Tang	22° Cotang	23° Tang	23° Cotang	′
0	.36397	2.74748	.38386	2.60509	.40403	2.47509	.42447	2.35585	60
1	.36430	2.74499	.38420	2.60283	.40436	2.47302	.42482	2.35395	59
2	.36463	2.74251	.38453	2.60057	.40470	2.47095	.42516	2.35205	58
3	.36496	2.74004	.38487	2.59831	.40504	2.46888	.42551	2.35015	57
4	.36529	2.73756	.38520	2.59606	.40538	2.46682	.42585	2.34825	56
5	.36562	2.73509	.38553	2.59381	.40572	2.46476	.42619	2.34636	55
6	.36595	2.73263	.38587	2.59156	.40606	2.46270	.42654	2.34447	54
7	.36628	2.73017	.38620	2.58932	.40640	2.46065	.42688	2.34258	53
8	.36661	2.72771	.38654	2.58708	.40674	2.45860	.42722	2.34069	52
9	.36694	2.72526	.38687	2.58484	.40707	2.45655	.42757	2.33881	51
10	.36727	2.72281	.38721	2.58261	.40741	2.45451	.42791	2.33693	50
11	.36760	2.72036	.38754	2.58038	.40775	2.45246	.42826	2.33505	49
12	.36793	2.71792	.38787	2.57815	.40809	2.45043	.42860	2.33317	48
13	.36826	2.71548	.38821	2.57593	.40843	2.44839	.42894	2.33130	47
14	.36859	2.71305	.38854	2.57371	.40877	2.44636	.42929	2.32943	46
15	.36892	2.71062	.38888	2.57150	.40911	2.44433	.42963	2.32756	45
16	.36925	2.70819	.38921	2.56928	.40945	2.44230	.42998	2.32570	44
17	.36958	2.70577	.38955	2.56707	.40979	2.44027	.43032	2.32383	43
18	.36991	2.70335	.38988	2.56487	.41013	2.43825	.43067	2.32197	42
19	.37024	2.70094	.39022	2.56266	.41047	2.43623	.43101	2.32012	41
20	.37057	2.69853	.39055	2.56046	.41081	2.43422	.43136	2.31826	40
21	.37090	2.69612	.39089	2.55827	.41115	2.43220	.43170	2.31641	39
22	.37123	2.69371	.39122	2.55608	.41149	2.43019	.43205	2.31456	38
23	.37157	2.69131	.39156	2.55389	.41183	2.42819	.43239	2.31271	37
24	.37190	2.68892	.39190	2.55170	.41217	2.42618	.43274	2.31086	36
25	.37223	2.68653	.39223	2.54952	.41251	2.42418	.43308	2.30902	35
26	.37256	2.68414	.39257	2.54734	.41285	2.42218	.43343	2.30718	34
27	.37289	2.68175	.39290	2.54516	.41319	2.42019	.43378	2.30534	33
28	.37322	2.67937	.39324	2.54299	.41353	2.41819	.43412	2.30351	32
29	.37355	2.67700	.39357	2.54082	.41387	2.41620	.43447	2.30167	31
30	.37388	2.67462	.39391	2.53865	.41421	2.41421	.43481	2.29984	30
31	.37422	2.67225	.39425	2.53648	.41455	2.41223	.43516	2.29801	29
32	.37455	2.66989	.39458	2.53432	.41490	2.41025	.43550	2.29619	28
33	.37488	2.66752	.39492	2.53217	.41524	2.40827	.43585	2.29437	27
34	.37521	2.66516	.39526	2.53001	.41558	2.40629	.43620	2.29254	26
35	.37554	2.66281	.39559	2.52786	.41592	2.40432	.43654	2.29073	25
36	.37588	2.66046	.39593	2.52571	.41626	2.40235	.43689	2.28891	24
37	.37621	2.65811	.39626	2.52357	.41660	2.40038	.43724	2.28710	23
38	.37654	2.65576	.39660	2.52142	.41694	2.39841	.43758	2.28528	22
39	.37687	2.65342	.39694	2.51929	.41728	2.39645	.43793	2.28348	21
40	.37720	2.65109	.39727	2.51715	.41763	2.39449	.43828	2.28167	20
41	.37754	2.64875	.39761	2.51502	.41797	2.39253	.43862	2.27987	19
42	.37787	2.64642	.39795	2.51289	.41831	2.39058	.43897	2.27806	18
43	.37820	2.64410	.39829	2.51076	.41865	2.38863	.43932	2.27626	17
44	.37853	2.64177	.39862	2.50864	.41899	2.38668	.43966	2.27447	16
45	.37887	2.63945	.39896	2.50652	.41933	2.38473	.44001	2.27267	15
46	.37920	2.63714	.39930	2.50440	.41968	2.38279	.44036	2.27088	14
47	.37953	2.63483	.39963	2.50229	.42002	2.38084	.44071	2.26909	13
48	.37986	2.63252	.39997	2.50018	.42036	2.37891	.44105	2.26730	12
49	.38020	2.63021	.40031	2.49807	.42070	2.37697	.44140	2.26552	11
50	.38053	2.62791	.40065	2.49597	.42105	2.37504	.44175	2.26374	10
51	.38086	2.62561	.40098	2.49386	.42139	2.37311	.44210	2.26196	9
52	.38120	2.62332	.40132	2.49177	.42173	2.37118	.44244	2.26018	8
53	.38153	2.62103	.40166	2.48967	.42207	2.36925	.44279	2.25840	7
54	.38186	2.61874	.40200	2.48758	.42242	2.36733	.44314	2.25663	6
55	.38220	2.61646	.40234	2.48549	.42276	2.36541	.44349	2.25486	5
56	.38253	2.61418	.40267	2.48340	.42310	2.36349	.44384	2.25309	4
57	.38286	2.61190	.40301	2.48132	.42345	2.36158	.44418	2.25132	3
58	.38320	2.60963	.40335	2.47924	.42379	2.35967	.44453	2.24956	2
59	.38353	2.60736	.40369	2.47716	.42413	2.35776	.44488	2.24780	1
60	.38386	2.60509	.40403	2.47509	.42447	2.35585	.44523	2.24604	0
′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	′
	69°		68°		67°		66°		

′	24° Tang	24° Cotang	25° Tang	25° Cotang	26° Tang	26° Cotang	27° Tang	27° Cotang	′
0	.44523	2.24604	.46631	2.14451	.48773	2.05030	.50953	1.96261	60
1	.44558	2.24428	.46666	2.14288	.48809	2.04879	.50989	1.96120	59
2	.44593	2.24252	.46702	2.14125	.48845	2.04728	.51026	1.95979	58
3	.44627	2.24077	.46737	2.13963	.48881	2.04577	.51063	1.95838	57
4	.44662	2.23902	.46772	2.13801	.48917	2.04426	.51099	1.95698	56
5	.44697	2.23727	.46808	2.13639	.48953	2.04276	.51136	1.95557	55
6	.44732	2.23553	.46843	2.13477	.48989	2.04125	.51173	1.95417	54
7	.44767	2.23378	.46879	2.13316	.49026	2.03975	.51209	1.95277	53
8	.44802	2.23204	.46914	2.13154	.49062	2.03825	.51246	1.95137	52
9	.44837	2.23030	.46950	2.12993	.49098	2.03675	.51283	1.94997	51
10	.44872	2.22857	.46985	2.12832	.49134	2.03526	.51319	1.94858	50
11	.44907	2.22683	.47021	2.12671	.49170	2.03376	.51356	1.94718	49
12	.44942	2.22510	.47056	2.12511	.49206	2.03227	.51393	1.94579	48
13	.44977	2.22337	.47092	2.12350	.49242	2.03078	.51430	1.94440	47
14	.45012	2.22164	.47128	2.12190	.49278	2.02929	.51467	1.94301	46
15	.45047	2.21992	.47163	2.12030	.49315	2.02780	.51503	1.94162	45
16	.45082	2.21819	.47199	2.11871	.49351	2.02631	.51540	1.94023	44
17	.45117	2.21647	.47234	2.11711	.49387	2.02483	.51577	1.93885	43
18	.45152	2.21475	.47270	2.11552	.49423	2.02335	.51614	1.93746	42
19	.45187	2.21304	.47305	2.11392	.49459	2.02187	.51651	1.93608	41
20	.45222	2.21132	.47341	2.11233	.49495	2.02039	.51688	1.93470	40
21	.45257	2.20961	.47377	2.11075	.49532	2.01891	.51724	1.93332	39
22	.45292	2.20790	.47412	2.10916	.49568	2.01743	.51761	1.93195	38
23	.45327	2.20619	.47448	2.10758	.49604	2.01596	.51798	1.93057	37
24	.45362	2.20449	.47483	2.10600	.49640	2.01449	.51835	1.92920	36
25	.45397	2.20278	.47519	2.10442	.49677	2.01302	.51872	1.92782	35
26	.45432	2.20108	.47555	2.10284	.49713	2.01155	.51909	1.92645	34
27	.45467	2.19938	.47590	2.10126	.49749	2.01008	.51946	1.92508	33
28	.45502	2.19769	.47626	2.09969	.49786	2.00862	.51983	1.92371	32
29	.45538	2.19599	.47662	2.09811	.49822	2.00715	.52020	1.92235	31
30	.45573	2.19430	.47698	2.09654	.49858	2.00569	.52057	1.92098	30
31	.45608	2.19261	.47733	2.09498	.49894	2.00423	.52094	1.91962	29
32	.45643	2.19092	.47769	2.09341	.49931	2.00277	.52131	1.91826	28
33	.45678	2.18923	.47805	2.09184	.49967	2.00131	.52168	1.91690	27
34	.45713	2.18755	.47840	2.09028	.50004	1.99986	.52205	1.91554	26
35	.45748	2.18587	.47876	2.08872	.50040	1.99841	.52242	1.91418	25
36	.45784	2.18419	.47912	2.08716	.50076	1.99695	.52279	1.91282	24
37	.45819	2.18251	.47948	2.08560	.50113	1.99550	.52316	1.91147	23
38	.45854	2.18084	.47984	2.08405	.50149	1.99406	.52353	1.91012	22
39	.45889	2.17916	.48019	2.08250	.50185	1.99261	.52390	1.90876	21
40	.45924	2.17749	.48055	2.08094	.50222	1.99116	.52427	1.90741	20
41	.45960	2.17582	.48091	2.07939	.50258	1.98972	.52464	1.90607	19
42	.45995	2.17416	.48127	2.07785	.50295	1.98828	.52501	1.90472	18
43	.46030	2.17249	.48163	2.07630	.50331	1.98684	.52538	1.90337	17
44	.46065	2.17083	.48198	2.07476	.50368	1.98540	.52575	1.90203	16
45	.46101	2.16917	.48234	2.07321	.50404	1.98396	.52613	1.90069	15
46	.46136	2.16751	.48270	2.07167	.50441	1.98253	.52650	1.89935	14
47	.46171	2.16585	.48306	2.07014	.50477	1.98110	.52687	1.89801	13
48	.46206	2.16420	.48342	2.06860	.50514	1.97966	.52724	1.89667	12
49	.46242	2.16255	.48378	2.06706	.50550	1.97823	.52761	1.89533	11
50	.46277	2.16090	.48414	2.06553	.50587	1.97681	.52798	1.89400	10
51	.46312	2.15925	.48450	2.06400	.50623	1.97538	.52836	1.89266	9
52	.46348	2.15760	.48486	2.06247	.50660	1.97395	.52873	1.89133	8
53	.46383	2.15596	.48521	2.06094	.50696	1.97253	.52910	1.89000	7
54	.46418	2.15432	.48557	2.05942	.50733	1.97111	.52947	1.88867	6
55	.46454	2.15268	.48593	2.05790	.50769	1.96969	.52985	1.88734	5
56	.46489	2.15104	.48629	2.05637	.50806	1.96827	.53022	1.88602	4
57	.46525	2.14940	.48665	2.05485	.50843	1.96685	.53059	1.88469	3
58	.46560	2.14777	.48701	2.05333	.50879	1.96544	.53096	1.88337	2
59	.46595	2.14614	.48737	2.05182	.50916	1.96402	.53134	1.88205	1
60	.46631	2.14451	.48773	2.05030	.50953	1.96261	.53171	1.88073	0
′	65° Cotang	65° Tang	64° Cotang	64° Tang	63° Cotang	63° Tang	62° Cotang	62° Tang	′

′	28°	
	Tang	Cotang
0	.53171	1.88073
1	.53208	1.87941
2	.53246	1.87809
3	.53283	1.87677
4	.53320	1.87546
5	.53358	1.87415
6	.53395	1.87283
7	.53432	1.87152
8	.53470	1.87021
9	.53507	1.86891
10	.53545	1.86760
11	.53582	1.86630
12	.53620	1.86499
13	.53657	1.86369
14	.53694	1.86239
15	.53732	1.86109
16	.53769	1.85979
17	.53807	1.85850
18	.53844	1.85720
19	.53882	1.85591
20	.53920	1.85462
21	.53957	1.85333
22	.53995	1.85204
23	.54032	1.85075
24	.54070	1.84946
25	.54107	1.84818
26	.54145	1.84689
27	.54183	1.84561
28	.54220	1.84433
29	.54258	1.84305
30	.54296	1.84177
31	.54333	1.84049
32	.54371	1.83922
33	.54409	1.83794
34	.54446	1.83667
35	.54484	1.83540
36	.54522	1.83413
37	.54560	1.83286
38	.54597	1.83159
39	.54635	1.83033
40	.54673	1.82906
41	.54711	1.82780
42	.54748	1.82654
43	.54786	1.82528
44	.54824	1.82402
45	.54862	1.82276
46	.54900	1.82150
47	.54938	1.82025
48	.54975	1.81899
49	.55013	1.81774
50	.55051	1.81649
51	.55089	1.81524
52	.55127	1.81399
53	.55165	1.81274
54	.55203	1.81150
55	.55241	1.81025
56	.55279	1.80901
57	.55317	1.80777
58	.55355	1.80653
59	.55393	1.80529
60	.55431	1.80405
	Cotang	Tang
′	61°	

′	32° Tang	32° Cotang	33° Tang	33° Cotang	34° Tang	34° Cotang	35° Tang	35° Cotang	′
0	.62487	1.60033	.64941	1.53986	.67451	1.48256	.70021	1.42815	60
1	.62527	1.59930	.64982	1.53888	.67493	1.48163	.70064	1.42726	59
2	.62568	1.59826	.65024	1.53791	.67536	1.48070	.70107	1.42638	58
3	.62608	1.59723	.65065	1.53693	.67578	1.47977	.70151	1.42550	57
4	.62649	1.59620	.65106	1.53595	.67620	1.47885	.70194	1.42462	56
5	.62689	1.59517	.65148	1.53497	.67663	1.47792	.70238	1.42374	55
6	.62730	1.59414	.65189	1.53400	.67705	1.47699	.70281	1.42286	54
7	.62770	1.59311	.65231	1.53302	.67748	1.47607	.70325	1.42198	53
8	.62811	1.59208	.65272	1.53205	.67790	1.47514	.70368	1.42110	52
9	.62852	1.59105	.65314	1.53107	.67832	1.47422	.70412	1.42022	51
10	.62892	1.59002	.65355	1.53010	.67875	1.47330	.70455	1.41934	50
11	.62933	1.58900	.65397	1.52913	.67917	1.47238	.70499	1.41847	49
12	.62973	1.58797	.65438	1.52816	.67960	1.47146	.70542	1.41759	48
13	.63014	1.58695	.65480	1.52719	.68002	1.47053	.70586	1.41672	47
14	.63055	1.58593	.65521	1.52622	.68045	1.46962	.70629	1.41584	46
15	.63095	1.58490	.65563	1.52525	.68088	1.46870	.70673	1.41497	45
16	.63136	1.58388	.65604	1.52429	.68130	1.46778	.70717	1.41409	44
17	.63177	1.58286	.65646	1.52332	.68173	1.46686	.70760	1.41322	43
18	.63217	1.58184	.65688	1.52235	.68215	1.46595	.70804	1.41235	42
19	.63258	1.58083	.65729	1.52139	.68258	1.46503	.70848	1.41148	41
20	.63299	1.57981	.65771	1.52043	.68301	1.46411	.70891	1.41061	40
21	.63340	1.57879	.65813	1.51946	.68343	1.46320	.70935	1.40974	39
22	.63380	1.57778	.65854	1.51850	.68386	1.46229	.70979	1.40887	38
23	.63421	1.57676	.65896	1.51754	.68429	1.46137	.71023	1.40800	37
24	.63462	1.57575	.65938	1.51658	.68471	1.46046	.71066	1.40714	36
25	.63503	1.57474	.65980	1.51562	.68514	1.45955	.71110	1.40627	35
26	.63544	1.57372	.66021	1.51466	.68557	1.45864	.71154	1.40540	34
27	.63584	1.57271	.66063	1.51370	.68600	1.45773	.71198	1.40454	33
28	.63625	1.57170	.66105	1.51275	.68642	1.45682	.71242	1.40367	32
29	.63666	1.57069	.66147	1.51179	.68685	1.45592	.71285	1.40281	31
30	.63707	1.56969	.66189	1.51084	.68728	1.45501	.71329	1.40195	30
31	.63748	1.56868	.66230	1.50988	.68771	1.45410	.71373	1.40109	29
32	.63789	1.56767	.66272	1.50893	.68814	1.45320	.71417	1.40022	28
33	.63830	1.56667	.66314	1.50797	.68857	1.45229	.71461	1.39936	27
34	.63871	1.56566	.66356	1.50702	.68900	1.45139	.71505	1.39850	26
35	.63912	1.56466	.66398	1.50607	.68942	1.45049	.71549	1.39764	25
36	.63953	1.56366	.66440	1.50512	.68985	1.44958	.71593	1.39679	24
37	.63994	1.56265	.66482	1.50417	.69028	1.44868	.71637	1.39593	23
38	.64035	1.56165	.66524	1.50322	.69071	1.44778	.71681	1.39507	22
39	.64076	1.56065	.66566	1.50228	.69114	1.44688	.71725	1.39421	21
40	.64117	1.55966	.66608	1.50133	.69157	1.44598	.71769	1.39336	20
41	.64158	1.55866	.66650	1.50038	.69200	1.44508	.71813	1.39250	19
42	.64199	1.55766	.66692	1.49944	.69243	1.44418	.71857	1.39165	18
43	.64240	1.55666	.66734	1.49849	.69286	1.44329	.71901	1.39079	17
44	.64281	1.55567	.66776	1.49755	.69329	1.44239	.71946	1.38994	16
45	.64322	1.55467	.66818	1.49661	.69372	1.44149	.71990	1.38909	15
46	.64363	1.55368	.66860	1.49566	.69416	1.44060	.72034	1.38824	14
47	.64404	1.55269	.66902	1.49472	.69459	1.43970	.72078	1.38738	13
48	.64446	1.55170	.66944	1.49378	.69502	1.43881	.72122	1.38653	12
49	.64487	1.55071	.66986	1.49284	.69545	1.43792	.72167	1.38568	11
50	.64528	1.54972	.67028	1.49190	.69588	1.43703	.72211	1.38484	10
51	.64569	1.54873	.67071	1.49097	.69631	1.43614	.72255	1.38399	9
52	.64610	1.54774	.67113	1.49003	.69675	1.43525	.72299	1.38314	8
53	.64652	1.54675	.67155	1.48909	.69718	1.43436	.72344	1.38229	7
54	.64693	1.54576	.67197	1.48816	.69761	1.43347	.72388	1.38145	6
55	.64734	1.54478	.67239	1.48722	.69804	1.43258	.72432	1.38060	5
56	.64775	1.54379	.67282	1.48629	.69847	1.43169	.72477	1.37976	4
57	.64817	1.54281	.67324	1.48536	.69891	1.43080	.72521	1.37891	3
58	.64858	1.54183	.67366	1.48442	.69934	1.42992	.72565	1.37807	2
59	.64899	1.54085	.67409	1.48349	.69977	1.42903	.72610	1.37722	1
60	.64941	1.53986	.67451	1.48256	.70021	1.42815	.72654	1.37638	0
′	57° Cotang	57° Tang	56° Cotang	56° Tang	55° Cotang	55° Tang	54° Cotang	54° Tang	′

′	36° Tang	36° Cotang	37° Tang	37° Cotang	38° Tang	38° Cotang	39° Tang	39° Cotang	′
0	.72654	1.37638	.75355	1.32704	.78129	1.27994	.80978	1.23490	60
1	.72699	1.37554	.75401	1.32624	.78175	1.27917	.81027	1.23416	59
2	.72743	1.37470	.75447	1.32544	.78222	1.27841	.81075	1.23343	58
3	.72788	1.37386	.75492	1.32464	.78269	1.27764	.81123	1.23270	57
4	.72832	1.37302	.75538	1.32384	.78316	1.27688	.81171	1.23196	56
5	.72877	1.37218	.75584	1.32304	.78363	1.27611	.81220	1.23123	55
6	.72921	1.37134	.75629	1.32224	.78410	1.27535	.81268	1.23050	54
7	.72966	1.37050	.75675	1.32144	.78457	1.27458	.81316	1.22977	53
8	.73010	1.36967	.75721	1.32064	.78504	1.27382	.81364	1.22904	52
9	.73055	1.36883	.75767	1.31984	.78551	1.27306	.81413	1.22831	51
10	.73100	1.36800	.75812	1.31904	.78598	1.27230	.81461	1.22758	50
11	.73144	1.36716	.75858	1.31825	.78645	1.27153	.81510	1.22685	49
12	.73189	1.36633	.75904	1.31745	.78692	1.27077	.81558	1.22612	48
13	.73234	1.36549	.75950	1.31666	.78739	1.27001	.81606	1.22539	47
14	.73278	1.36466	.75996	1.31586	.78786	1.26925	.81655	1.22467	46
15	.73323	1.36383	.76042	1.31507	.78834	1.26849	.81703	1.22394	45
16	.73368	1.36300	.76088	1.31427	.78881	1.26774	.81752	1.22321	44
17	.73413	1.36217	.76134	1.31348	.78928	1.26698	.81800	1.22249	43
18	.73457	1.36134	.76180	1.31269	.78975	1.26622	.81849	1.22176	42
19	.73502	1.36051	.76226	1.31190	.79022	1.26546	.81898	1.22104	41
20	.73547	1.35968	.76272	1.31110	.79070	1.26471	.81946	1.22031	40
21	.73592	1.35885	.76318	1.31031	.79117	1.26395	.81995	1.21959	39
22	.73637	1.35802	.76364	1.30952	.79164	1.26319	.82044	1.21886	38
23	.73681	1.35719	.76410	1.30873	.79212	1.26244	.82092	1.21814	37
24	.73726	1.35637	.76456	1.30795	.79259	1.26169	.82141	1.21742	36
25	.73771	1.35554	.76502	1.30716	.79306	1.26093	.82190	1.21670	35
26	.73816	1.35472	.76548	1.30637	.79354	1.26018	.82238	1.21598	34
27	.73861	1.35389	.76594	1.30558	.79401	1.25943	.82287	1.21526	33
28	.73906	1.35307	.76640	1.30480	.79449	1.25867	.82336	1.21454	32
29	.73951	1.35224	.76686	1.30401	.79496	1.25792	.82385	1.21382	31
30	.73996	1.35142	.76733	1.30323	.79544	1.25717	.82434	1.21310	30
31	.74041	1.35060	.76779	1.30244	.79591	1.25642	.82483	1.21238	29
32	.74086	1.34978	.76825	1.30166	.79639	1.25567	.82531	1.21166	28
33	.74131	1.34896	.76871	1.30087	.79686	1.25492	.82580	1.21094	27
34	.74176	1.34814	.76918	1.30009	.79734	1.25417	.82629	1.21023	26
35	.74221	1.34732	.76964	1.29931	.79781	1.25343	.82678	1.20951	25
36	.74267	1.34650	.77010	1.29853	.79829	1.25268	.82727	1.20879	24
37	.74312	1.34568	.77057	1.29775	.79877	1.25193	.82776	1.20808	23
38	.74357	1.34487	.77103	1.29696	.79924	1.25118	.82825	1.20736	22
39	.74402	1.34405	.77149	1.29618	.79972	1.25044	.82874	1.20665	21
40	.74447	1.34323	.77196	1.29541	.80020	1.24969	.82923	1.20593	20
41	.74492	1.34242	.77242	1.29463	.80067	1.24895	.82972	1.20522	19
42	.74538	1.34160	.77289	1.29385	.80115	1.24820	.83022	1.20451	18
43	.74583	1.34079	.77335	1.29307	.80163	1.24746	.83071	1.20379	17
44	.74628	1.33998	.77382	1.29229	.80211	1.24672	.83120	1.20308	16
45	.74674	1.33916	.77428	1.29152	.80258	1.24597	.83169	1.20237	15
46	.74719	1.33835	.77475	1.29074	.80306	1.24523	.83218	1.20166	14
47	.74764	1.33754	.77521	1.28997	.80354	1.24449	.83268	1.20095	13
48	.74810	1.33673	.77568	1.28919	.80402	1.24375	.83317	1.20024	12
49	.74855	1.33592	.77615	1.28842	.80450	1.24301	.83366	1.19953	11
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51	.74946	1.33430	.77708	1.28687	.80546	1.24153	.83465	1.19811	9
52	.74991	1.33349	.77754	1.28610	.80594	1.24079	.83514	1.19740	8
53	.75037	1.33268	.77801	1.28533	.80642	1.24005	.83564	1.19669	7
54	.75082	1.33187	.77848	1.28456	.80690	1.23931	.83613	1.19599	6
55	.75128	1.33107	.77895	1.28379	.80738	1.23858	.83662	1.19528	5
56	.75173	1.33026	.77941	1.28302	.80786	1.23784	.83712	1.19457	4
57	.75219	1.32946	.77988	1.28225	.80834	1.23710	.83761	1.19387	3
58	.75264	1.32865	.78035	1.28148	.80882	1.23637	.83811	1.19316	2
59	.75310	1.32785	.78082	1.28071	.80930	1.23563	.83860	1.19246	1
60	.75355	1.32704	.78129	1.27994	.80978	1.23490	.83910	1.19175	0
′	Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	′
	53°		52°		51°		50°		

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Cotang	Tang	Cotang	Tang	Cotang	Tang	Cotang	Tang	′
49°		48°		47°		46°		

PART II.

Strength of Materials, and Stability of Structures.

INTRODUCTION.

the chapters constituting this part of the book, the author ıdeavored to present to architects and builders handy and e rules and tables for determining the strength or stability of ece of work they may have in hand. Every pains has been to present the rules in the simplest form consistent with accuracy; and it is believed that all constants and theories ced are fully up to the knowledge of the present day, some constants on transverse strength having but recently been ıined. The rules for wrought-iron columns have lately been y changed by some engineers; but as the question of the th of wrought-iron columns has not yet been satisfactorily l, and as the formulas herein given undoubtedly err on the ide if at all, we have thought best not to change them, espeas they are still used by many bridge engineers.

question of the wind-pressure on roofs has not been taken as thorough manner as would be needed for pitch roofs of reat span; but for ordinary wooden roofs, and iron roofs not ling one hundred feet span, the method given in Chap. I. is sufficiently accurate.

one wishing to study the most accurate method of obtaining fect of the wind-pressure on roofs will find it in Professor 's excellent work on "Graphical Analysis of Roof Trusses."

onclusion, the author recommends these chapters as presentcurate and modern rules, especially adapted to the requireof American practice.

EXPLANATION OF SIGNS AND TERMS USED IN THE FOLLOWING FORMULAS.

Besides the usual arithmetical signs and characters in general use, the following characters and abbreviations will frequently be used: —

The sign $\sqrt{\ }$ means square root of number behind.

$\sqrt[3]{\ }$ means cube root of number behind.

( ) means that all the numbers between are to be taken as one quantity.

. means decimal parts; $2.5 = 2\frac{5}{10}$, or $.46 = \frac{46}{100}$.

The letter A denotes the co-efficient of strength for beams one inch square, and one foot between the supports.

C denotes resistance, in pounds, of a block of any material to crushing, per square inch of section.

E denotes the modulus of elasticity of any material, in pounds per square inch.

e denotes constant for stiffness of beams.

F denotes resistance of any material to shearing, per square inch.

R denotes the modulus of rupture of any material.

S denotes a factor of safety.

T denotes resistance of any material to being pulled apart, in pounds, per square inch of cross-section.

Breadth is used to denote the least side of a rectangular piece, and is always measured in inches.

Depth denotes the vertical height of a beam or girder, and is always to be taken in inches, unless expressly stated otherwise.

Length denotes the distance between supports *in feet*, unless otherwise specified.

Abbreviations. — In order to shorten the formulas, it has often been found necessary to use certain abbreviations; such as bet. for between, bot. for bottom, dist. for distance, diam. for diameter, hor. for horizontal, sq. for square, etc., which, however, can in no case lead to uncertainty as to their meaning.

Where the word "ton" is used in this volume, it always means 2000 pounds.

CHAPTER I.

DEFINITIONS OF TERMS USED IN MECHANICS.

THE following terms frequently occur in treating of mechanical construction, and it is essential that their meaning be well understood.

Mechanics is the science which treats of the action of forces.

Applied Mechanics treats of the laws of mechanics which relate to works of human art; such as beams, trusses, arches, etc.

Rest is the relation between two points, when the straight line joining them does not change in length or direction.

A body is at rest relatively to a point, when any point in the body is at rest relatively to the first-mentioned point.

Motion is the relation between two points, when the straight line joining them changes in length or direction, or in both.

A body moves relatively to a point, when any point in the body moves relatively to the point first mentioned.

Force is that which changes, or tends to change, the state of a body in reference to rest or motion. It is a cause regarding the essential nature of which we are ignorant. We cannot deal with forces properly, but only with the laws of their action.

Equilibrium is that condition of a body in which the forces acting upon it balance or neutralize each other.

Statics is that part of Applied Mechanics which treats of the conditions of equilibrium, and is divided into: —

a. Statics of rigid bodies.

b. Hydrostatics.

In building we have to deal only with the former.

Structures are artificial constructions in which all the parts are intended to be in equilibrium and at rest, as in the case of a bridge or roof-truss.

They consist of two or more solid bodies, called **pieces,** which are connected at portions of their surfaces called **joints.**

There are three conditions of equilibrium in a structure; viz.: —

I. The forces exerted on the whole structure must balance each other. These forces are: —

a. The weight of the structure.

b. The load it carries.

c. The supporting pressures, or resistance of the foundations, called external forces.

II. The forces exerted on each piece must balance each other. These forces are: —

a. The weight of the piece.

b. The load it carries.

c. The resistance of its joints.

III. The forces exerted on each of the parts into which any piece may be supposed to be divided must balance each other.

Stability consists in the fulfilment of conditions I. and II., that is, the ability of the structure to resist displacement of its parts.

Strength consists in the fulfilment of condition III., that is, the ability of a piece to resist breaking.

Stiffness consists in the ability of a piece to resist bending.

The theory of structures is divided into two parts; viz.: —

I. That which treats of strength and stiffness, dealing only with single pieces, and generally known as **strength of materials.**

II. That which treats of stability, dealing with structures.

Stress. — The load or system of forces acting on any piece of material is often denoted by the term "stress," and the word will be so used in the following pages.

The *intensity of the stress* per square inch on any normal surface of a solid is the total stress divided by the area of the section in square inches. Thus, if we had a bar ten feet long and two inches square, with a load of 8000 pounds pulling in the direction of its length, the stress on any normal section of the rod would be 8000 pounds; and the intensity of the stress per square inch would be $8000 \div 4$, or 2000 pounds.

Strain. — When a solid body is subjected to any kind of stress, an alteration is produced in the volume and figure of the body, and this alteration is called the "strain." In the case of the bar given above, the strain would be the amount that the bar would stretch under its load.

The **Ultimate Strength,** or **Breaking Load,** of a body is the load required to produce fracture in some specified way.

The **Safe Load** is the load that a piece can support without impairing its strength.

Factors of Safety. — When not otherwise specified, a *factor of safety* means the ratio in which the breaking load exceeds the safe load. In designing a piece of material to sustain a certain load, it is required that it shall be perfectly safe under all circumstances; and hence it is necessary to make an allowance for any defects in the material, workmanship, etc. It is obvious, that, for

ls of different composition, different factors of safety will ired. Thus, iron being more homogeneous than wood, and ole to defects, it does not require so great a factor of safety. gain, different kinds of strains require different factors of Thus, a long wooden column or strut requires a greater of safety than a wooden beam. As the factors thus vary erent kinds of strains and materials, we will give the proper of safety for the different strains when considering the ce of the material to those strains.

inction between Dead and Live Load.—The dead load," as used in mechanics, means a load that is ap- y imperceptible degrees, and that remains steady; such as ght of the structure itself.

ive load" is one that is applied suddenly, or accompanied brations; such as swift trains travelling over a railway- or a force exerted in a moving machine.

s been found by experience, that the effect of a live load on or other piece of material is twice as severe as that of a ad of the same weight: hence a piece of material designed r a live load should have a factor of safety *twice* as large lesigned to carry a dead load.

load produced by a crowd of people walking on a floor is considered to produce an effect which is a mean between a dead and live load, and a factor of safety is adopted ngly.

Modulus of Rupture is a constant quantity found in nulas for strength of iron beams, and is eighteen times the the constant "A."

ulus of Elasticity.—If we take a bar of any elastic l, one inch square, and of any length, secured at one end, the other apply a force pulling in the direction of its length, find by careful measurement that the bar has been stretched gated by the action of the force.

if we divide the total elongation in inches by the original of the bar in inches, we shall have the elongation of the bar t of length; and, if we divide the pulling-force per square this latter quantity, we shall have what is known as the s of elasticity.

e we may define the *modulus of elasticity as the pulling or ssing force per unit of section divided by the elongation pression per unit of length.*

n example of the method of determining the modulus of y of any material, we will take the following:—

ose we have a bar of wrought-iron, two inches square and

ten feet long, securely fastened at one end, and to the other end we apply a pulling-force of 40,000 pounds. This force causes the bar to stretch, and by careful measurement we find the elongation to be 0.0414 of an inch. Now, as the bar is ten feet, or 120 inches, long, if we divide 0.0414 by 120, we shall have the elongation of the bar per unit of length.

Performing this operation, we have as the result 0.00034 of an inch. As the bar is two inches square, the area of cross-section is four square inches, and hence the pulling-force per square inch is 10,000 pounds. Then, dividing 10,000 by 0.00034, we have as the modulus of elasticity of the bar 29,400,000 pounds.

This is the method generally employed to determine the modulus of elasticity of iron ties; but it can also be obtained from the deflection of beams, and it is in that way that the values of the modulus for most woods have been found.

Another definition of the modulus of elasticity, and which is a natural consequence of the one just given, is the number of pounds that would be required to stretch or shorten a bar one inch square by an amount equal to its length, provided that the law of perfect elasticity would hold good for so great a range. The modulus of elasticity is generally denoted by E, and is used in the determination of the stiffness of beams.

Moment. — If we take any solid body, and pivot it at any point, and apply a force to the body, acting in any direction except in a line with the pivot, we shall produce rotation of the body, provided the force is sufficiently strong. This rotation is produced by what is called the *moment* of the force; and *the moment of a force* about any given point or pivot is the product of the force into the perpendicular distance from the pivot to the line of action of the force, or, in common phrase, *the product of the force into the arm with which it acts.*

The Centre of Gravity of a body is the point through which the resultant of the weight of the body always acts, no matter in what position the body be. If a body be suspended at its centre of gravity, and revolved in any direction, it will always be in equilibrium.

(For centre of gravity of surfaces, lines, and solids, see Chap. IV.)

CLASSIFICATION OF STRAINS WHICH MAY BE PRODUCED IN A SOLID BODY.

The different strains to which building-materials may be exposed are: —

I. **Tension,** as in the case of a weight suspended from one end of a rod, rope, tie-bar, etc.; the other end being fixed, tending to stretch or lengthen the fibres.

II. **Shearing Strain,** as in the case of treenails, pins in bridges, etc., where equal forces are applied on opposite sides in such a manner as to tend to force one part over the adjacent one.

III. **Compression,** as in the case of a weight resting on top of a column or post, tending to compress the fibres.

IV. **Transverse** or **Cross Strain,** as in the case of a load on a beam, tending to bend it.

V. **Torsion,** a twisting strain, which seldom occurs in building-construction, though quite frequently in machinery.

CHAPTER II.

FOUNDATIONS.

THE following chapter on Foundations is intended to furnish the reader with only a general knowledge of the subject, and to enable him to be sure that he is within the limits of safety if he follows what is here given. For foundations of large works, or buildings upon soil of questionable firmness, the compressibility of the soil should be determined by experiments.

The term "foundation" is used to designate all that portion of any structure which serves only as a basis on which to erect the superstructure.

This term is sometimes applied to that portion of the solid material of the earth upon which the structure rests, and also to the artificial arrangements which may be made to support the base.

In the following pages these will be designated by the term "foundation-bed."

Object of Foundations. — The object to be obtained in the construction of any foundation is to form such a solid base for the superstructure that no movement shall take place after its erection. But all structures built of coarse masonry, whether of stone, or brick, will settle to a certain extent; and, with a few exceptions, all soils will become compressed under the weight of almost any building.

Our main object, therefore, is not to prevent settlement entirely, but to insure that it shall be uniform; so that, after the structure is finished, it will have no cracks or flaws, however irregularly it may be disposed over the area of its site.

Foundations Classed. — Foundations may be divided into two classes: —

CLASS I. — *Foundations constructed in situations where the natural soil is sufficiently firm to bear the weight of the intended structure.*

CLASS II. — *Foundations in situations where an artificial bearing-stratum must be formed, in consequence of the softness or looseness of the soil.*

Each of these two great classes may be subdivided into two divisions: —

a. Foundations in situations where water offers no impediment to the execution of the work.

b. Foundations under water.

It is seldom that architects design buildings whose foundations are under water; and, as this division of the subject enters rather deeply into the science of engineering, we shall not discuss it here.

Boring. — Before we can decide what kind of foundation it will be necessary to build, we must know the nature of the subsoil. If not already known, this is determined, ordinarily, by digging a trench, or making a pit, close to the site of the proposed works, to a depth sufficient to allow the different strata to be seen.

For important structures, the nature of the subsoil is often determined by boring with the tools usually employed for this purpose. When this method is employed, the different kinds and thickness of the strata are determined by examining the specimens brought up by the auger used in boring.

Foundations of the First Class. — The foundations included under this class may be divided into two cases, according to the different kinds of soil on which the foundation is to be built: —

CASE I. — *Foundations on soil composed of materials whose stability is not affected by saturation with water, and which are firm enough to support the weight of the structure.*

Under this case belong, —

Foundations on Rock. — To prepare a rock foundation for being built upon, all that is generally required is to cut away the loose and decayed portions of the rock, and to dress the rock to a plane surface as nearly perpendicular to the direction of the pressure as is practicable; or, if the rock forms an inclined plane, to cut a series of plane surfaces, like those of steps, for the wall to rest on. If there are any fissures in the rock, they should be filled with concrete or rubble masonry. Concrete is better for this purpose, as, when once set, it is nearly incompressible under any thing short of a crushing-force; so that it forms a base almost as solid as the rock itself, while the compression of the mortar joints of the masonry is certain to cause some irregular settlement.

If it is unavoidably necessary that some parts of the foundation shall start from a lower level than others, care should be taken to keep the mortar joints as close as possible, or to execute the lower portions of the work in cement, or some hard-setting mortar: otherwise the foundations will settle unequally, and thus cause much injury to the superstructure. The load placed on the rock should at no time exceed one-eighth of that necessary to crush it. Pro-

fessor Rankine gives the following examples of the actual intensity of the pressure per square foot on some existing rock foundations:—

Average of ordinary cases, the rock being at least as strong as the strongest red bricks 20000
Pressures at the base of St. Rollox chimney (450 feet below the summit)——
On a layer of strong concrete or beton, 6 feet deep 6670
On sandstone below the beton, so soft that it crumbles in the hand 4000

The last example shows the pressure which is safely borne in practice by one of the weakest substances to which the name of rock can be applied.

M. Jules Gaudard, C.E., states, that, on a rocky ground, the Roquefavour aqueduct exerts a pressure of 26,800 pounds to the square foot. A bed of solid rock is unyielding, and appears at first sight to offer all the advantages of a secure foundation. It is generally found in practice, however, that, in large buildings, part of the foundations will not rest on the rock, but on the adjacent soil; and as the soil, of whatever material it may be composed, is sure to be compressed somewhat, irregular settlement will almost invariably take place, and give much trouble. The only remedy in such a case is to make the bed for the foundation resting on the soil as firm as possible, and lay the wall, to the level of the rock, in cement or hard-setting mortar.

Foundation on Compact Stony Earths, such as Gravel or Sand. —Strong gravel may be considered as one of the best soils to build upon; as it is almost incompressible, is not affected by exposure to the atmosphere, and is easily levelled.

Sand is also almost incompressible, and forms an excellent foundation as long as it can be kept from escaping; but as it has no cohesion, and acts like a fluid when exposed to running water, it should be treated with great caution.

The foundation bed in soils of this kind is prepared by digging a trench from four to six feet deep, so that the foundation may be started below the reach of the disintegrating effects of frost.

The bottom of the trench is levelled; and, if parts of it are required to be at different levels, it is broken into steps.

Care should be taken to keep the surface-water from running into the trench; and, if necessary, drains should be made at the bottom to carry away the water.

The weight resting on the bottom of the trench should be proportional to the resistance of the material forming the bed.

Mr. Gaudard says that a load of 16,500 to 18,300 pounds per square foot has been put upon close sand in the foundations of Gorai Bridge, and on gravel in the Lock Ken Viaduct at Bordeaux.

In the bridge at Nantes, there is a load of 15,200 pounds to the square foot on sand; but some settlement has already taken place.

Rankine gives the greatest intensity of pressure on foundations in firm earth at from 2500 to 3500 pounds per square foot.

In order to distribute the pressure arising from the weight of the structure over a greater surface, it is usual to give additional breadth to the foundation courses: this increase of breadth is called the spread. In compact, strong earth, the spread is made one and a half times the thickness of the wall, and, in ordinary earth or sand, twice that thickness.

CASE II. — *Foundations on soils firm enough to support the weight of the structure, but whose stability is affected by water.*

The principal soil under this class, with which we have to do, is a clay soil.

In this soil the bed is prepared by digging a trench, as in rocky soils; and the foundation must be sure to start below the frost-line, for the effect of frost in clay soils is very great.

The soil is also much affected by the action of water; and hence the ground should be well drained before the work is begun, and the trenches so arranged that the water shall not remain in them. And, in general, the less a soil of this kind is exposed to the air and weather, and the sooner it is protected from exposure, the better for the work. In building on a clay bank, great caution should be used to secure thorough drainage, that the clay may not have a tendency to slide during wet weather.

The safe load for stiff clay and marl is given by Mr. Gaudard at from 5500 to 11,000 pounds per square foot. Under the cylindrical piers of the Szegedin Bridge in Hungary, the soil, consisting of clay intermixed with fine sand, bears a load of 13,300 pounds to the square foot; but it was deemed expedient to increase its supporting power by driving some piles in the interior of the cylinder, and also to protect the cylinder by sheeting outside.

Mr. McAlpine, M. Inst. C.E., in building a high wall at Albany, N.Y., succeeded in safely loading a wet clay soil with two tons to the square foot, but with a settlement depending on the depth of the excavation. In order to prevent a great influx of water, and consequent softening of the soil, he surrounded the excavation with a puddle trench ten feet high and four feet wide, and he also spread a layer of coarse gravel on the bottom.

Foundations in Soft Earths. — There are three materials in general use for forming an artificial bearing-stratum in soft soils.

Whichever material is employed, the bed is first prepared by excavating a trench sufficiently deep to place the foundation-courses below the action of frost and rain. Great caution should be used in cases of this kind to prevent unequal settling.

The bottom of the trench is made level, and covered with a bed of stones, sand, or concrete.

Stones. — When stone is used, the bottom of the trench should be paved with rubble or cobble stones, well settled in place by ramming. On this paving, a bed of concrete is then laid.

Sand. — In all situations where the ground, although soft, is of sufficient consistency to confine the sand, this material may be used with many advantages as regards both the cost and the stability of the work. The quality which sand possesses, of distributing the pressure put upon it, in both a horizontal and vertical direction, makes it especially valuable for a foundation bed in this kind of soil; as the lateral pressure exerted against the sides of the foundation pit greatly relieves the bottom.

There are two methods of using sand; viz., in layers and as piles. In forming a *stratum* of sand, it is spread in layers of about nine inches in thickness, and each layer well rammed before the next one is spread. The total depth of sand used should be sufficient to admit of the pressure on the upper surface of the sand being distributed over the entire bottom of the trench.

Sand-piling is a very economical and efficient method of forming a foundation under some circumstances. It would not, however, be effective in very loose, wet soils; as the sand would work into the surrounding ground.

Sand-piling is executed by making holes in the soil, or in the bottom of the trench, about six or seven inches in diameter, and about six feet deep, and filling them with damp sand, well rammed so as to force it into every cavity.

In situations where the stability of piles arises from the pressure of the ground around them, sand-piles are found of more service than timber ones, for the reason that the timber-pile transmits pressure only in a vertical direction, while the sand-pile transmits it over the whole surface of the hole it fills, thus acting on a large area of bearing-surface. The ground above the piles should be covered with planking, concrete, or masonry, to prevent its being forced up by the lateral pressure exerted by the piles; and, on the stratum thus formed, the foundation walls may be built in the usual manner.

Foundations on Piles. — Where the soil upon which we wish to build is not firm enough to support the foundation, one of the most common methods of forming a solid foundation bed is

by driving wooden piles into the soil, and placing the foundation walls upon these.

The piles are generally round, and have a length of about twenty times their mean diameter of cross-section. The diameter of the head varies from nine to eighteen inches. The piles should be straight grained, and free from knots and ring strokes. Fir, beach, oak, and Florida yellow-pine are the best woods for piles; though spruce and hemlock are very commonly used.

Where piles are exposed to tide-water, they are generally driven with their bark on. In other cases, it is not essential.

Piles which are driven through hard ground, generally require to have an iron hoop fixed tightly on their heads to prevent them from splitting, and also to be *shod* with iron shoes, either of cast or wrought iron.

Long piles may be divided into two classes, — those which transmit the load to a firm soil, thus acting as pillars; and those where the pile and its load are wholly supported by the friction of the earth on the sides of the pile.

In order to ascertain the safe load which it will do to put upon a pile of the first class, it is only necessary to calculate the safe crushing-strength of the wood; but, for piles of the second and more common class, it is not so easy to determine the maximum load which they will safely support.

Many writers have endeavored to give rules for calculating the effect of a given blow in sinking a pile; but investigations of this kind are of little practical value, because we can never be in possession of sufficient data to obtain even an approximate result. The effect of each blow on the pile will depend on the momentum of the blow, the velocity of the ram, the relative weights of the ram and the pile, the elasticity of the pile-head, and the resistance offered by the ground through which the pile is passing; and, as the last-named conditions cannot well be ascertained, any calculations in which they are only assumed must of necessity be mere guesses.

Load on Piles. — Professor Rankine gives the limits of the safe load on piles, based upon practical examples, as follows: —

For piles driven till they reach the firm ground, 1000 pounds per square inch of area of head.

For piles standing in soft ground by friction, 200 pounds per square inch of area of head.

But as, in the latter case, so much depends upon the character of the soil in which the piles are driven, such a general rule as the above is hardly to be recommended.

Several rules for the bearing-load on piles have been given,

Perhaps the rule most commonly given is that of Major Sanders, United-States Engineer. He experimented largely at Fort Delaware, and in 1851 gave the following rule as reliable for ordinary pile-driving.

Sanders's Rule for determining the load for a common wooden pile, driven until it sinks through only small and nearly equal distances under successive blows:—

$$\text{Safe load in lbs.} = \frac{\text{weight of hammer in lbs.} \times \text{fall in inches}}{8 \times \text{sinking at last blow}}.$$

Mr. John C. Trautwine, C.E., in his pocket-book for engineers, gives a rule which appears to agree very well with actual results.

His rule is expressed as follows:—

$$\begin{array}{c}\text{Extreme load in} \\ \text{tons of 2240 lbs.}\end{array} = \frac{\begin{array}{c}\text{cube root of} \\ \text{fall in feet}\end{array} \times \begin{array}{c}\text{weight of} \\ \text{hammer in lbs.}\end{array} \times 0.023}{\text{Last sinking in inches} + 1}.$$

For the safe load he recommends that one-half the extreme load should be taken for piles thoroughly driven in firm soils, and one-fourth when driven in river-mud or marsh.

According to Mr. Trautwine, the French engineers consider a pile safe for a load of 25 tons when it refuses to sink under a hammer of 1344 pounds falling 4 feet.

The test of a pile having been sufficiently driven, according to the best authorities, is that it shall not sink more than one-fifth of an inch under thirty blows of a ram weighing 800 pounds, falling 5 feet at each blow.

A more common rule is to consider the pile fully driven when it does not sink more than one-fourth of an inch at the last blow of a ram weighing 2500 pounds, falling 30 feet.

In ordinary pile-driving for buildings, however, the piles often sink more than this at the last blow; but, as the piles are seldom loaded to their full capacity, it is not necessary to be so particular as in the foundations of engineering structures. A common practice with architects is to specify the length of the piles to be used, and the piles are driven until their heads are just above ground, and then left to be levelled off afterwards.

Example of Pile Foundation.—As an example of the method of determining the necessary number of piles to support a given building, we will determine the number of piles required to support the side-walls of a warehouse (of which a vertical sec-

tion is shown in Fig. 1). The walls are of brick, and the weight may be taken at 110 pounds per cubic foot of masonry.

The piles are to be driven in two rows, two feet on centres; and it is found that a pile 20 feet long and 10 inches at the top will sink

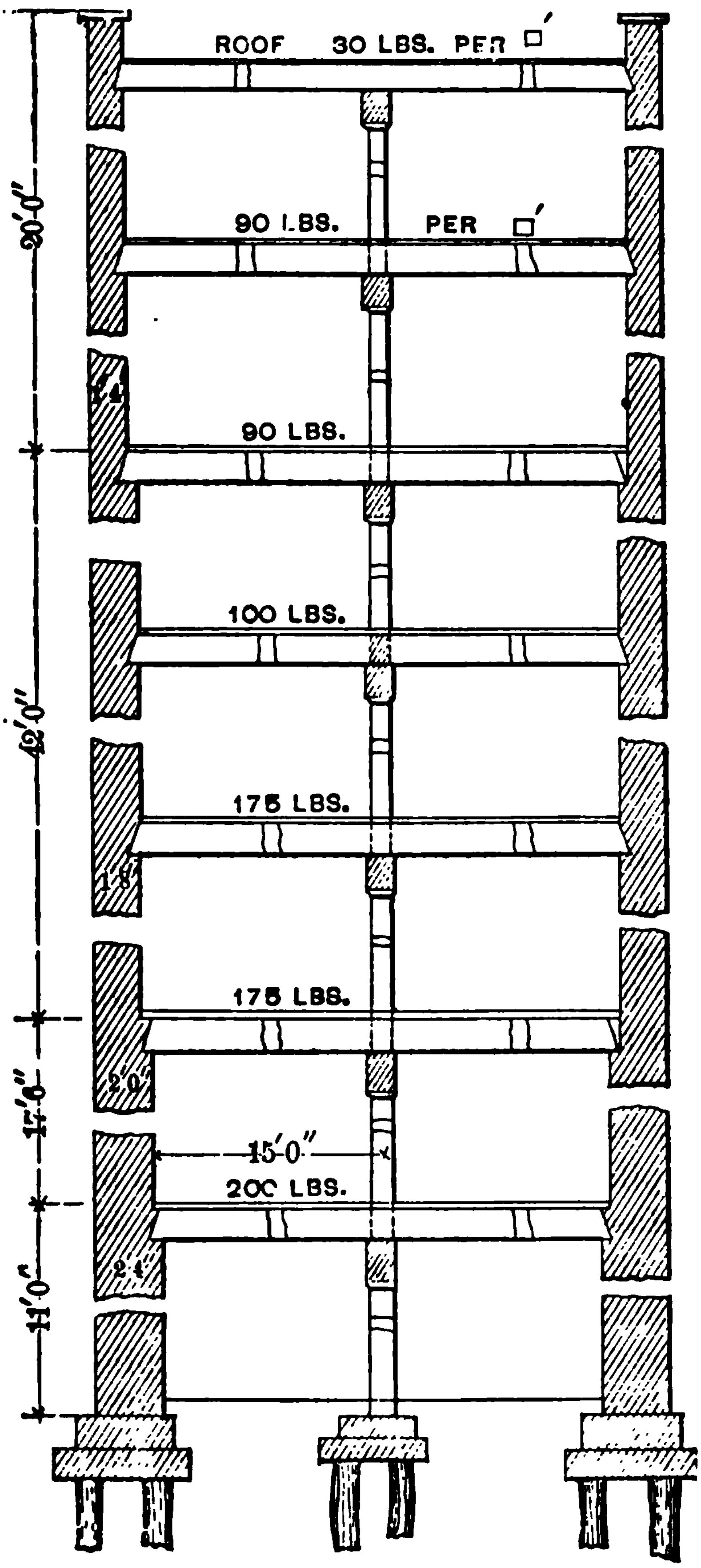

Fig. 1.

one inch under a 1200-pound hammer falling 20 feet after the pile has been entirely driven into the soil. What distance should the piles be on centres lengthwise of the wall?

By calculation we find that the wall contains $157\frac{1}{3}$ cubic feet of masonry per running foot, and hence weighs 17,306 pounds.

The load from the floors which comes upon the wall is:—

From the first floor	1500 lbs.
From the second floor	1380 lbs.
From the third floor	1380 lbs.
From the fourth floor	790 lbs.
From the fifth floor	720 lbs.
From the sixth floor	720 lbs.
From the roof	240 lbs.
Total	6730 lbs.

Hence the total weight of the wall and its load per running foot is 24,036 pounds.

The load which one of the piles will support is, by Sanders's rule, $\frac{1200 \times 240}{8 \times 1} = 36000$ pounds.

By Trautwine's rule, using a factor of safety of 2.5, the safe load would be

$$\frac{\sqrt[3]{20} \times 1200 \times 0.023}{2.5 \times (1 + 1)} = 15 \text{ tons (of 2240 pounds), or 33600 pounds.}$$

Then one *pair* of piles would support 72,000, or 67,200 pounds, according to which rule we take.

Dividing these numbers by the weight of one foot of the wall and its load, we find, that, by Sanders's rule, one pair of piles will support 3 feet of the wall, and, by Trautwine's rule, 2.8 feet of wall: hence the piles should be placed 2 feet 9 inches or 3 feet on centres.

In very heavy buildings, heavy timbers are sometimes bolted to the tops of the piles, and the foundation walls built on these.

In Boston, Mass., a large part of the city is built upon made land, and hence the buildings have to be supported by pile foundations. The Building Laws of the city require that all buildings "exceeding thirty-five feet in height (with pile foundation) shall have not less than two rows of piles under all external and party walls, and the piles shall be spaced not over three feet on centres in the direction of the length of the wall."

As an example of the load which ordinary piles in the made land of Boston will support, it may be stated that the piles under Trinity Church in Boston support two tons each, approximately.

For engineering works, various kinds of iron piles are used; but they are too rarely used for foundations of buildings to come within the scope of this chapter. For a description of these

ıe reader should consult some standard work on engineering. good description of iron piles is given in "Wheeler's Civil ›ering," and also in "Trautwine's Handbook."

ıcrete Foundation Beds.—Concrete is largely used ndation beds in soft soil, and is a very valuable material for rpose; as it affords a firm solid bed, and can be spread out › distribute the pressure over a large area.

:rete is an artificial compound, generally made by mixing cement with sand, water, and some hard material, as broken slag, bits of brick, earthenware, burnt clay, shingle, etc. e is any choice of the materials forming the base of the :e, the preference should be given to fragments of a some-›orous nature, such as pieces of brick or limestone, rather › those with smooth surfaces. (*See page* 148*a*.)

broken material used in the concrete is sometimes, for con-:e, called the *aggregate*, and the mortar in which it is incased, ıtrix. The aggregate is generally broken so as to pass h a $1\frac{1}{2}$ or 2 inch mesh.

ımp ground or under water, hydraulic lime should of course l in mixing the concrete.

ing Concrete.—A very common practice in laying con-: to tip the concrete, after mixing, from a height of six or ›et into the trench where it is to be deposited. This process :ted to by the best authorities, on the ground that the heavy ;ht portions separate while falling, and that the concrete is ›re not uniform throughout its mass.

best method is to wheel the concrete in barrows, immedi-fter mixing, to the place where it is to be laid, gently tipping position, and carefully ramming into layers about twelve thick. After each layer has been allowed to set, it should pt clean, wetted, and made rough, by means of a pick, for the yer.

: contractors make the concrete courses the exact width d, keeping up the sides with boards, if the trench is too This is a bad practice; for when the sides of the founda-:s are carefully trimmed, and the concrete rammed up solidly them, the concrete is less liable to be crushed and broken it has entirely consolidated. It is therefore desirable that ›cifications for concrete work should require that the whole of the excavation be filled, and that, if the trenches are ted too wide, the extra amount of concrete be furnished at ıtractor's expense.

:rete made with hydraulic lime is sometimes designated as or beton

The pressure allowed on a concrete bed should not exceed one-tenth part of its resistance to crushing. Trautwine gives as the average crushing-strength of concrete forty tons per square foot.

Foundations in Compressible Soil.—The great difficulty met with in forming a firm bed in compressible soils arises from the nature of the soil, and its yielding in all directions under pressure. (*See page* 144.)

There are several methods which have been successfully employed in soils of this kind.

I. When the compressible material is of a moderate depth, the excavation is made to extend to the firm soil beneath, and the foundation put in, as in firm soils.

The principal objection to this method is the expense, which would often be very great.

II. A second method is to drive piles through the soft soil into the firm soil beneath. The piles are then cut off at a given level and a timber platform laid upon the top of the piles, which serves as a support for the foundation, and also ties the tops of the piles together.

III. A modification of the latter method is to use shorter piles, which are only driven in the compressible soil. The platform is made to extend over so large an area that the intensity of the pressure per square foot is within the safe limits for this particular soil.

IV. Another modification of the second method consists in using piles of only five or six inches in diameter, and only five or six feet long, and placing them as near together as they can be driven. A platform of timber is then placed on the piles, as in the second method.

The object of the short piles is to compress the soil, and make it firmer. "This practice is one not to be recommended; its effect being usually to pound up the soil, and to bring it into a state which can best be described by comparing it to batter-pudding."[1]

V. Still another method is to surround the site of the work with sheet-piling (flat piles driven close together, so as to form a sheet), to prevent the escape of the soil, which is then consolidated by driving piles into it at short distances from each other. The piles are then sawn off level, and the ground excavated between them for two or three feet, and filled up with concrete: the whole is then planked over to receive the superstructure.

The great point to be attended to in building foundations in soils of this kind is to distribute the weight of the structure equally

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[1] Dobson on Foundations.

over the foundation, which will then settle in a vertical direction, and cause little injury; whereas any irregular settlement would rend the work from top to bottom.

Planking for Foundation Beds. — In erecting buildings on soft ground, where a large bearing-surface is required, planking may be resorted to with great advantage, provided the timber can be kept from decay. If the ground is wet and the timber good, there is little to fear in this respect; but in a dry situation, or one exposed to alternations of wet and dry, no dependence can be placed on unprepared timber. There are several methods employed for the preservation of timber, such as kyanizing or creosoting; and the timber used for foundations should be treated by one of these methods.

The advantage of timber is, that it will resist a great cross-strain with very trifling flexure; and therefore a wide footing may be obtained without any excessive spreading of the bottom courses of the masonry. The best method of employing planking under walls is to cut the stuff into short lengths, which should be placed *across* the foundation, and tied longitudinally by planking laid to the width of the bottom course of masonry in the direction of the length of the wall, and firmly spiked to the bottom planking. Another good method of using planking is to lay down sleepers on the ground, and fill to their top with cement, and then place the planking on the level surface thus formed. For the cross-timbers, four-inch by six-inch timber, laid flatwise, will answer in ordinary cases.

Foundations for Chimneys. — As examples of the foundations required for very high chimneys, we quote the following from a treatise on foundations, in the latter part of a work on "Foundations and Foundation Walls," by George T. Powell.

Fig. 2.

Fig. 2 represents the base of a chimney erected in 1859 for the Chicago Refining Company, 151 feet high, and 12 feet square at the

foot. The base, merely two courses of heavy dimension stone, as shown, is bedded upon the surface-gravel near the mouth of the river, there recently deposited by the lake. The mortar employed in the joint between the stone is roofing-gravel in cement. The area of the base is 256 square feet, the weight of chimney, inclusive of base, 625 tons, giving a pressure of 34 pounds to the square inch. This foundation proved to be perfect.

Fig. 3 represents the base of a chimney erected in 1872 for the McCormick Reaper Works, Chicago, which is 160 feet high, 14 feet square at the foot, with a round flue of 6 feet 8 inches diameter.

Fig. 3.

The base covers 625 square feet; the weight of the chimney and base is approximately 1100 tons; the pressure upon the ground (dry hard clay) is therefore 24½ pounds to the square inch. This foundation also proved to be perfect in every respect.

Bearing Power of Soils.

(*Added to Ninth Edition.*)

In a paper published in the *American Architect and Building News*, November 3, 1888, by Prof Ira O. Baker, C.E., on the Bearing Power of Soils, he sums up the results of his discussion in the following table, which gives values which seem to the writer to be both practical and reliable. The remarks following the table should also be carefully considered.

Kind of Material.	Bearing power in tons per square foot.	
	Min.	Max.
Rock—the hardest—in thick layers, in native bed.....	200	
Rock equal to best ashlar masonry	25	30
Rock equal to best brick masonry	15	20
Rock equal to poor brick masonry	5	10
Clay on thick beds, always dry	4	6
Clay on thick beds, moderately dry....................	2	4
Clay, soft	1	2
Gravel and coarse sand, well cemented	8	10
Sand, compact and well cemented....................	4	6
Sand, clean, dry	2	4
Quicksand, alluvial soils, etc	0.5	1

"*Conclusion.*—It is well to notice that there are some practical considerations which modify the pressure which may safely be put upon the soil. For example, the pressure on the foundation of a tall chimney should be considerably less than that of the low massive foundation of a fireproof vault. In the former case a slight inequality of bearing power, and consequent unequal settling, might endanger the stability of the structure; while in the latter no serious harm would result. The pressure per unit of area should be less for a light structure subject to the passage of heavy loads—as, for example, a railroad viaduct—than for a heavy structure, subject only to a quiescent load, since the shock and jar of the moving load are far more serious than the heavier quiescent load."

The following list of actual known weight on different soils will give a very good idea of what has been done in actual practice.

Rock.—St. Rollox chimney, poorest kind of sandstone, 2 tons per square foot.

Clay.—Chimney, McCormick Reaper Works, Chicago, 1¾ tons per square foot on dry, hard clay.

Capitol at Albany, N. Y., rests on blue clay containing from 60 to 90 per cent. of alumina, the remainder being fine sand, and containing 40 per cent. of water on an average. The safe load was taken at 2 tons per square foot.

In the case of the Congressional Library at Washington, which rests on "yellow clay mixed with sand," 2½ tons per square foot was taken for the safe load. "Experience in Central Illinois shows that if the foundation is carried down below the action of the frost the clay subsoil will bear 1½ to 2 tons per square foot without appreciable settling." *

* Ira O. Baker, American Architect, November 3, 1888.

Sand and Gravel.—"In an experiment in France, clean river sand, compacted in a trench, supported 100 tons per square foot.

"The piers of the Cincinnati suspension bridge are founded on a bed of coarse gravel 12 feet below water; the maximum pressure on the gravel is 4 tons per square foot.

"The piers of the Brooklyn suspension bridge are founded 44 feet below the bed of the river, upon a layer of sand 2 feet thick resting upon bed-rock; the maximum pressure is about 5¼ tons per square foot.

"At Chicago, sand and gravel about 15 feet below the surface are successfully loaded with 2 to 2½ tons per square foot.

"At Berlin the safe load for sandy soil is generally taken at 2 to 2½ tons per square foot.

"The Washington Monument, Washington, D. C., rests upon a bed of very fine sand 2 feet thick. The ordinary pressure on certain parts of the foundation being not far from 11 tons per square foot, which the wind may increase to nearly 14 tons per square foot."*

Foundations on Soft, Yielding Soil, Built of Steel Beams and Concrete.—On page 141 is described the method of planking for foundations, which does very well where the timber is sure to be always wet, but, if there is any chance of its ever becoming dry, iron or steel beams should be used instead. Steel rails were first used embedded in concrete, but they offer, however, comparatively little resistance to deflection, and for this reason, if allowed to project beyond the masonry to any considerable length, the concrete filling is liable to crack, and thus the strength of the foundation become impaired.

Steel I-beams, more recently used for this purpose, are found to be superior in every respect. A greater depth can be adopted, the deflection thus reduced to a minimum and a sufficient saving effected to more than compensate for their additional cost per pound.

The foundation should be prepared (see illustration, p. 146) by first laying a bed of concrete to a depth of from 4 to 12 inches and then placing upon this a row of I-beams at right angles to the face of the wall. In the case of heavy piers, the beams may be crossed in two directions. Their distances apart, from centre to centre, may vary from 9 to 24 inches according to circumstances, *i.e.*, length of their projection beyond the masonry, thickness of concrete, estimated pressure per square foot, etc. They should be placed at least far enough apart to permit the introduction of the concrete

* Ira O. Baker, American Architect, November 3, 1888.

filling and its proper tamping between the beams. Unless the concrete is of unusual thickness, it will not be advisable to exceed 20 inches spacing, since otherwise the concrete may not be of sufficient strength to properly transmit the upward pressure to the beams. The most useful application of this method of founding is in localities where a thin and comparatively compact stratum overlies another of a more yielding nature. By using steel beams in such cases, the requisite spread at the base may be obtained without either penetrating the firm upper stratum or carrying the footing-courses to such a height as to encroach unduly upon the basement-room.

METHOD OF CALCULATING THE SIZE AND LENGTH OF THE BEAMS.*

Let L = Weight of wall per lineal foot, in tons.
and b = Assumed bearing capacity of ground, per square foot (usually from 1 to 3 tons).
Then $\frac{L}{b} = W$ = Required width of foundation, in feet.
w = Width of lowest course of footing stones.
p = Projection of beams beyond masonry, in feet.
s = Spacing of beams centre to centre, in feet.

Evidently the size of beams required will depend upon their strength as cantilevers of a length p, sustaining the upward reaction, which may be regarded as a uniformly distributed load.

Thus $p\,b$ = uniformly distributed load (in tons) on cantilevers, per lineal foot of wall,
and $p\,b\,s$ = uniform load in tons, on each beam.

The table on the following page gives the safe lengths p for the various sizes and weights of steel beams, for $s = 1$ foot and b ranging from 1 to 5 tons per square foot. For other values of s say 15 inches, *i. e.*, $1\frac{1}{4}$ feet, the table may be used by simply considering b increased in the same ratio as s (see example below). As regards the weight of beams, it is advantageous to assign to s as great a value as is warranted by the other considerations which obtain.

EXAMPLE SHOWING APPLICATION OF TABLE.

The weight of a brick wall, together with the load it must support, is 40 tons per lineal foot. The width of the lowest footing-course of masonry is 6 feet. Allowing a pressure of 2 tons per

* This and the next page are taken by permission from Carnegie, Phipps & Co.'s Pocket-book.

square foot on the foundation, what size and length of steel I-beams 18 inches centre to centre will be required?

Ans.: $L = 40$; $b = 2$; $w = 6$; $s = 1\frac{1}{2}$.

Therefore $W = 40 \div 2 = 20$ feet, the required length of beams. The projection $p = \frac{1}{2}(20 - 6) = 7$ feet.

In order to apply the table (calculated for $s = 1$ foot) we must consider b increased in the same ratio as s, *i.e.*, $b = 2 \times 1\frac{1}{2} = 3$ tons.

In the column for 3 tons, we find the length 7 feet to agree with 20 inches I-beams 64.0 pounds per foot.

TABLE GIVING SAFE LENGTHS OF PROJECTIONS p IN FEET (SEE ILLUSTRATION), FOR $s = 1$ FOOT AND VALUES OF b RANGING FROM 1 TO 5 TONS

Depth of beam, in.	Weight per foot lbs.	b (Tons per Square Foot). 1	1¼	1½
20	80	14.0	12.5	11.5
20	64	12.5	11.0	10.0
15	75	11.5	10.5	9.5
15	60	10.5	9.5	8.5
15	50	9.5	8.5	8.0
15	41	8.5	8.0	7.0
12	40	8.0	7.0	6.5
12	32	7.0	6.5	5.5
10	31	6.5	6.0	5.5
10	25.5	5.5	5.0	4.5
9	27	5.5	5.0	4.5
9	21	5.0	4.5	4.0
8	22	5.0	4.5	4.0
8	18	4.5	4.0	3.5
7	20	4.5	4.0	3.5
7	15.5	4.0	3.5	3.0
6	16	3.5	3.0	3.0
6	13	3.0	3.0	2.5
5	13	3.0	2.5	2.5
5	10	2.5	2.5	2.0
4	10	2.5	2.0	2.0
4	7.5	2.0	2.0	1.5

The foregoing table applies to *steel* beams. Values given based on extreme fibre strains of 16,000 pounds per square inch.

Chicago Foundations.*—The architects and builders of Chicago probably have to deal with the most unfavorable conditions for securing a good foundation for their heavy buildings of any people in the world.

The soil under the central part of the city consists of a black loamy clay, which is tolerably firm at the surface, and will sustain a load of from one to three tons per foot, depending upon locality. A few feet below the natural surface of the ground the soil becomes quite soft, growing more and more so the deeper the excavation is carried, and at a depth of from 12 to 18 feet it is so yielding that nothing can be placed upon it with any reliance. Nor is this all. It has been discovered, by many failures in buildings, that there is a broad subterranean layer of soft mud which lies directly across the most heavily built portion of the city, extending under the Post-office, and reaching from the lake to the river, a distance of three-quarters of a mile.

The first of the larger structures were built with continuous foundation walls, with wide footings, the width being proportioned to the loads bearing upon them. This method, however, did not prove successful, as it was found that the wall will settle more than a pier, and the corners of the wall will settle less than the centre.

After experiments of one kind and another, it has come to be the accepted practice in Chicago of dividing the foundation into isolated piers, the footing of each pier being carefully proportioned according to the load upon it, its position in the building, character of the superstructure, etc., so that all shall settle at exactly the same rate without any crackings or detriment to the superstructure.

The footings of the piers are built of steel beams and concrete, as described on page 145, except that the beams are often crossed three and four times; in this way a great spreading is obtained in a small height.

In determining the area of the footings, the ground is assumed to be capable of sustaining a safe load of from $1\frac{1}{2}$ to $2\frac{1}{4}$ tons per square foot. The loads on the piers of the Board of Trade building vary from $2\frac{3}{4}$ to $3\frac{7}{10}$ tons per square foot. The size of the footings under the piers and the corners is made less than under the walls, to offset the difference in settlement of the different portions of the building.

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* C. H. Blackall, in American Architect, p. 147, Vol. XXIII.

It is found that a heavy pier will sink proportionally more than a light one, so that the area under the larger piers is made relatively greater than under the smaller ones.

Again, it is necessary to take into account the material of which the superstructure is to be built. Thus, a footing under a brick wall is made larger than a footing under a line of iron columns, so that if both footings are loaded with the same weight, that under the columns will settle the most, to allow for the compression in the joints of the mason-work.

It is impossible to build heavy buildings on the Chicago soil without settlement, and the architect must therefore plan his building so that all parts shall settle equally, and this has been successfully done in many of the largest buildings.

In a building where the footings are proportioned to give a bearing weight on the ground of 2½ tons per square foot, it is estimated that the building will settle about 4 inches altogether.

Piling has been successfully used under several buildings in Chicago, and there seems to be no reason why it should not be more extensively resorted to.

In the construction of the large grain elevators which are scattered through the city the loads are so excessive, reaching as high as six tons per foot, that it would be impracticable to support them on ordinary footings, and piling has been resorted to. The piles are driven a distance of twenty to forty feet down to hard-pan, capped by wooden sleepers, with heavy wooden cross-beams and solid planking to receive the masonry.

CONCRETE FOOTING FOR FOUNDATIONS.

For the footings of foundations in nearly all kinds of soil where piles are not used, the writer believes a good concrete to be preferable to even the best dimension stone, for the reason that it acts as one piece of masonry and not as individual blocks of stone, and if made of sufficient thickness it will possess sufficient transverse strength to span any weak place in the soil beneath, if not of large area.

When the best brands of Portland cement are used, the proportions may be as follows :

One part Portland cement ; 3 parts clean sharp sand ; 5 parts chip stone, in sizes not exceeding 2 × 1½ × 3 inches. Using these proportions, one barrel of cement will make from 22 to 26 cubic feet of concrete.

The above proportions were used in the concrete for the foundations of the Mutual Life Insurance Company's Building, New York City

When the cement is not of the best quality, or other cement than Portland cement is used, more cement should be used with the other material. Using a cement made in the West, the author specifies that one part of cement to two of sand and four of broken stone should be used, and the result has been very satisfactory.

It will generally be found wise to keep an inspector constantly on the ground while the concrete is being put in, as the temptation to the contractor to economize on the cement is very great.

In mixing the concrete, the stone, sand, and cement should be thrown into the mortar box in the order named, and while one man turns on the water two or more men should rapidly and thoroughly work the material back and forth with shovels, when it should be immediately carried to the trenches. The concrete should be deposited in layers not over six inches thick, and each layer well rammed. If one layer dries before the next is deposited it should be well wet on top, just before depositing the next layer.

Care should be exercised to see that the trenches are not dug wider than the desired width of the footings ; and also in mixing the concrete, not to use more water than is necessary to bring the mass to a pudding-like consistency, as otherwise the cement may be washed away.

COST OF CONCRETE.

The cost of labor in mixing concrete, when the proper facilities are provided, need not exceed three cents a cubic foot, and four cents is a liberal allowance, with wages at two dollars a day. The amount of materials required to make 100 cubic feet of concrete may be taken as follows: proportion of 1 to 6, 5 bbls. cement (original package) and 4.4 yards of stone and sand; proportion of 1 to 8, 3.9 bbls. of cement and 4½ yards of aggregates.

The cost of concrete at the present time in Denver is about thirty cents per cubic foot.

The weight of concrete varies from 130 to 140 lbs. per cubic foot, according to the material used, granite aggregates making naturally the heaviest concrete.

CHAPTER III.

MASONRY WALLS.

Footing Courses.—In commencing the foundation walls of a building, it is customary to spread the bottom courses of the masonry considerably beyond the face of the wall, whatever be the character of the foundation bed, unless, perhaps, it be a solid rock bed, in which case the spreading of the walls would be useless. These spread courses are technically known as "footing courses." They answer two important purposes:—

1st, By distributing the weight of the structure over a larger area of bearing-surface, the liability to vertical settlement from the compression of the ground is greatly diminished.

2d, By increasing the area of the base of the wall, they add to its stability, and form a protection against the danger of the work being thrown out of "plumb" by any forces that may act against it.

Footings, to have any useful effect, must be securely bonded into the body of the work, and have sufficient strength to resist the violent cross-strains to which they are exposed.

Footings of Stone Foundations.—As, the lower any stone is placed in a building, the greater the weight it has to support and the risk arising from any defects in the laying and dressing of the stone, the footing courses should be of strong stone laid *on bed*, with the upper and lower faces dressed true. By laying *on bed* is meant laying the stone the same way that it lay before quarrying.

In laying the footing courses, no back joints should be allowed beyond the face of the upper work, except where the footings are in double courses; and every stone should bond into the body of the work several inches at least. Unless this is attended to, the footings will not receive the weight of the superstructure, and will be useless, as is shown in Fig. 1.

In proportion to the weight of the superstructure, the projection of each footing course beyond the one above it must be reduced, or the cross-strain thrown on the projecting portion of the masonry will rend it from top to bottom, as shown in Fig. 2.

In building large masses of work, such as the abutments of

bridges and the like, the proportionate increase of bearing-surface obtained by the footings is very slight, and there is generally great risk of the latter being broken off by the settlement of the body

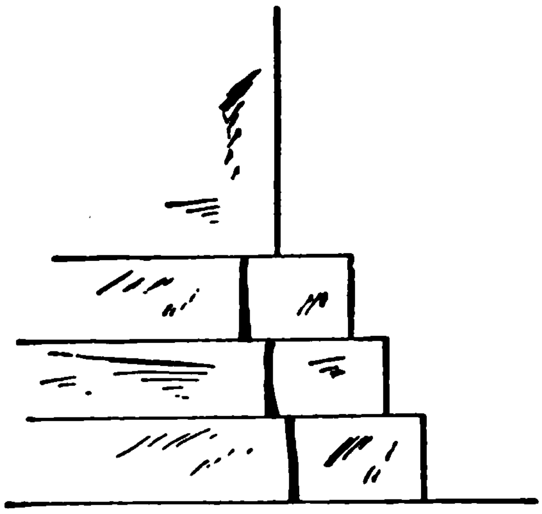


Fig. 1.

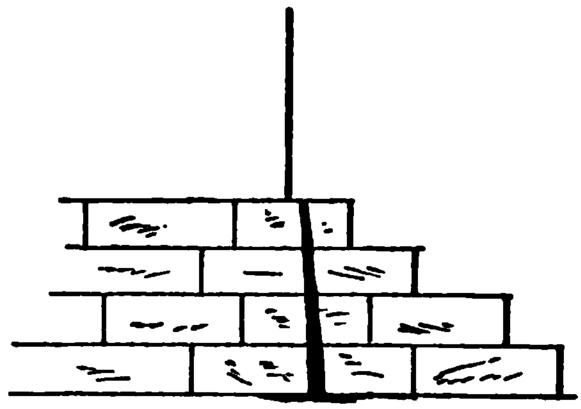


Fig. 2.

of the work, as in Fig. 2. It is therefore usual in these cases to give very little projection to the footing courses, and to bring up the work with a battering-face, or with a succession of very slight offsets, as in Fig. 3.

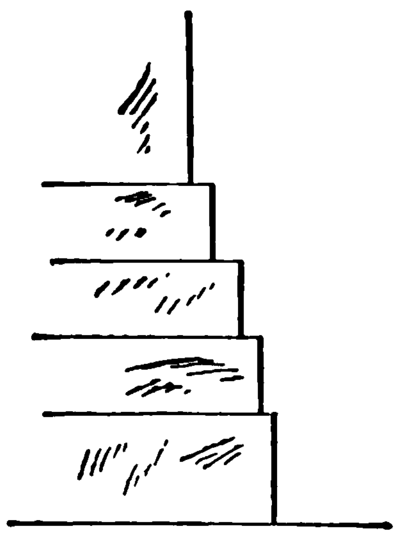


Fig. 3.

Footings of undressed rubble built in common mortar should never be used for buildings of any importance, as the compression of the mortar is sure to cause movements in the superstructure. If rubble must be used, it should be laid with cement mortar, so that the whole will form a solid mass; in which case the size and shape of the stone are of little consequence.

In general, footing stones should be at least two by three feet on the bottom, and eight inches thick.

The Building Laws of the city of New York require that the footing under all foundation walls, and under all piers, columns, posts, or pillars resting on the earth, shall be of stone or concrete. Under a foundation wall the footing must be at least twelve inches wider than the bottom width of the wall, and under piers, columns,

its, or pillars, at least twelve inches wider on all sides than the :tom width of the piers, columns, posts, or pillars, and not less ın eighteen inches in thickness; and, if built of stone, the stones ıll not be less than two by three feet, and at least eight inches ck.

All base-stones shall be well bedded, and laid edge to edge; and, the walls are built of isolated piers, then there must be inverted hes, at least twelve inches thick, turned under and between the rs, or two footing courses of large stone, at least ten inches ck in each course.

The Boston Building Laws require that the bottom course for all ındation walls resting upon the ground shall be at least twelve hes wider than the thickness given for the foundation walls.

Footings of Brick Foundations. — In building with ck, the special point to be attended to in the footing courses is

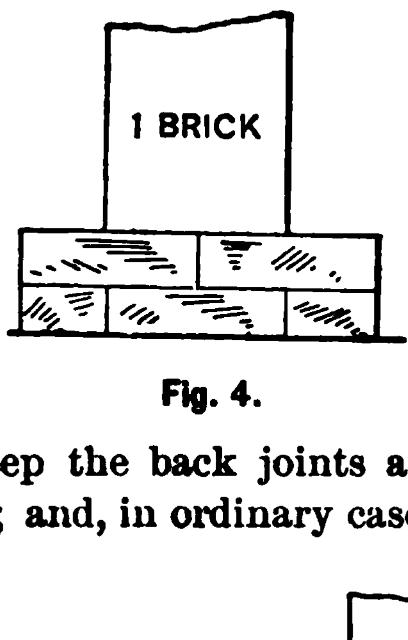


Fig. 4.

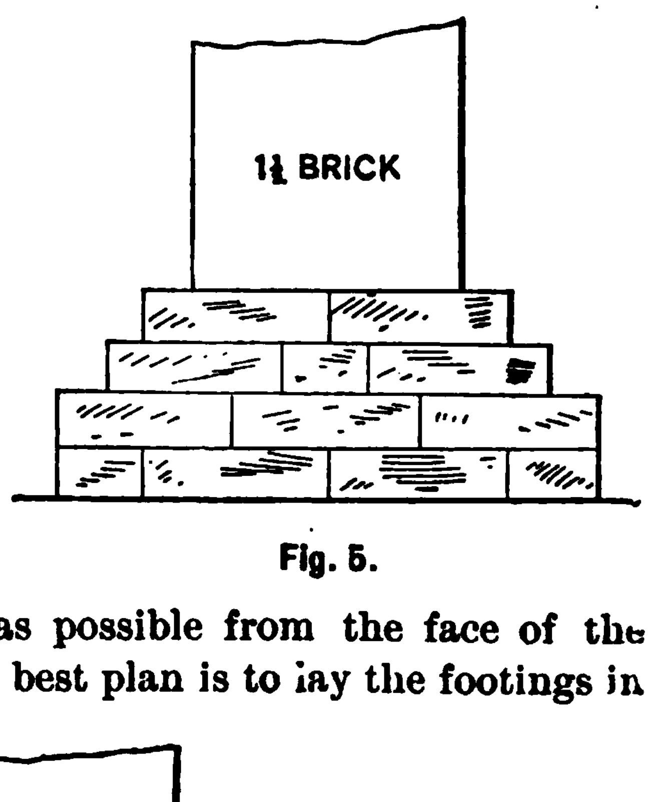


Fig. 5.

keep the back joints as far as possible from the face of the rk; and, in ordinary cases, the best plan is to lay the footings in

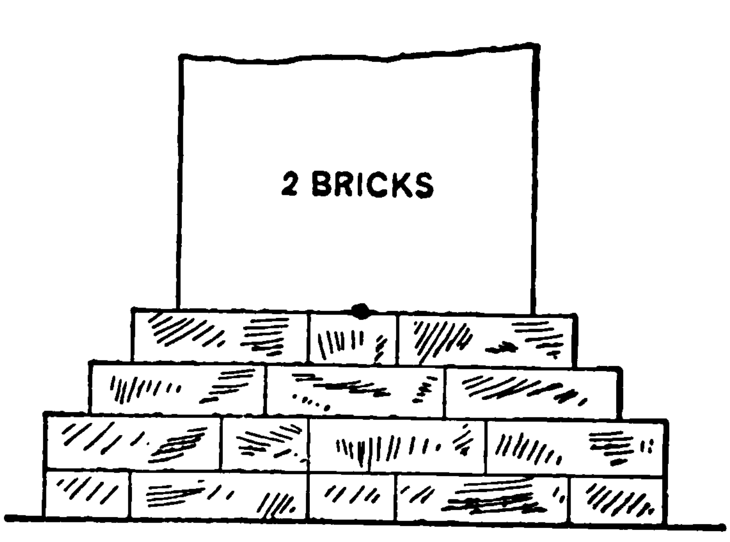


Fig. 6.

gle courses; the outside of the work being laid all headers, and course projecting more than one-fourth brick beyond the one ve it, except in the case of an eight-inch wall.

inches thick below the top floor, and stone walls not less than sixteen inches.

The thickness of the walls required by the laws of the cities of Boston, New York, and Denver, Colo., are shown by the tables on pp. 155-157.

The Boston Law also contains the following provisions, which form an excellent guide to architects in other localities:

SECTION 38. *Vaulted walls* shall contain, exclusive of withes, the same amount of material as is required for solid walls, and the walls on either side of the air-space shall be not less than eight inches thick, and shall be securely tied together with ties not more than two feet apart.

SECTION 39. In reckoning the thickness of walls, no allowance shall be made for *ashlar,* unless it is eight inches or more thick, in which case the excess over four inches shall be reckoned as part of the thickness of the wall. Ashlar shall be at least four inches thick, and properly held by metal clamps to the backing, or properly bonded to the same.

SECTION 40. External walls may be built in part of *iron or steel,* and when so built may be of less thickness than is above required for external walls, provided such walls meet the requirements of this act as to strength, and provided that all constructional parts are wholly protected from heat by brick or terra-cotta, or by plastering three-quarters of an inch thick, with iron furring and wiring.

FIRST AND SECOND CLASS BUILDINGS.

SECTION 45. First and second class buildings hereafter built shall have floor bearing supports not over thirty feet apart. These supports may be brick walls, trusses or columns and girders. Such brick walls may be four inches less in thickness than is required by this act for external and party walls of the same height, provided they comply with the provisions of this act as to the strength of materials, but in no case less than twelve inches thick. When trusses are used, the walls upon which they rest shall be at least four inches thicker than is otherwise required by sections thirty-six and thirty-seven, for every addition of twenty-five feet or part thereof to the length of the truss over thirty feet.

SECTION 46. Second class buildings hereafter built shall be so divided by brick partition walls of the thickness prescribed for bearing partition walls and carried twelve inches above the roof, that no space inside any such building shall exceed in area ten thousand square feet, and no existing wall in any second class

building shall be removed so as to leave an area not so enclosed, of more than ten thousand square feet.

SECTION 47. All walls of a first or second class building meeting at an angle shall be united every ten feet of their height, by anchors made of at least two inches by half an inch wrought iron securely built in to the side or partition walls not less than thirty-six inches, and into the front and rear walls at least one-half the thickness of such walls.

The New York Law also provides that the bearing walls of all buildings exceeding one hundred and five feet in depth without a cross wall, or piers or buttresses, shall be increased four inches in thickness for each additional one hundred and five feet in depth or part thereof; also, in case the walls of any building are less than twenty feet apart and less than forty feet in depth, or there are cross walls, or piers or buttresses, which serve to strengthen the walls, the thickness of the interior walls may be reduced in thickness at the judgment of the superintendent of buildings. In comparing the thickness of brick walls in the eastern and western portions of the country, it should be taken into consideration that the eastern brick are much harder and stronger than those in the west, and that an eight-inch wall in Boston is probably as strong (to resist crushing) as a thirteen-inch wall in Denver, Colo.

THICKNESS OF WALLS REQUIRED IN DENVER, COLO.

FOR DWELLINGS, TENEMENTS, OR LODGING HOUSES.

Outside and Party Walls.	Basement.	1st Story.	2d Story.	3d Story.	4th Story.	5th Story.	6th Story.
One story	13	8					
Two stories	13	13	8				
Three stories	17	13	13	13			
Four stories	22	17	13	13	13		
Five stories	22	17	17	13	13	13	
Six stories	26	22	22	17	13	13	13

BUILDINGS OTHER THAN THE ABOVE.

One story	13	8					
Two stories	17	13	13				
Three stories	22	17	13	13			
Four stories	26	22	17	13	13		
Five stories	26	22	17	17	13	13	
Six stories	30	26	22	22	17	17	13

THICKNESS OF WALLS REQUIRED IN BOSTON (Laws of 1892).

DWELLING-HOUSES.

Height of Walls above Curb.	Foundations.		Outside and Party Walls.
	Granite.	Brick.	
Not exceeding 33 feet high, 20 feet wide, and 40 feet deep.	16″	12″	8 inches.
33 feet and not exceeding 60 feet.	20″	16″	12 inches.
60 feet and not exceeding 70 feet.	24″	20″	16″ to top of 2d floor, 12″ remaining height.
70 feet and not exceeding 80 feet.	28″	24″	20″ to top of 2d floor, 16″ to top of upper floor and to wit 15 feet of the roof, and 12″ the remaining height.

Walls exceeding 80 feet in height shall have, for the upper 80 feet, the thickness required for buildings between 70 and 80 feet in heig and every section of 25 feet or part thereof, below such upper 80 feet, shall have a thickness of 4″ more than is required for the sect next above it.

BUILDINGS OF THE FIRST AND SECOND CLASS, OTHER THAN DWELLINGS.

Height of Walls above Curb.	Granite.	Brick.	Outside and Party Walls.
40 feet or less	24″	20″	16″ to top of 2d floor, and 12″ the remaining height.
40 feet and not exceeding 60 feet.	28″	24″	20″ to top of 2d floor, and 16″ the remaining height.
60 feet and not exceeding 80 feet.	32″	28″	24″ to top of 1st floor, 20″ to the top of the upper floor and within 15 feet of the roof, and 16″ above.

Walls exceeding 80 feet in height shall have, for the upper 80 feet, the thickness required for buildings between 60 and 80 feet in heig and every section of 25 feet or part thereof, below such upper 80 feet, shall have a thickness of 4″ more than is required for the sect next above it.

THICKNESS OF WALLS REQUIRED IN THE CITY OF NEW YORK (Laws of 1882).

DWELLING-HOUSES, APARTMENT-HOUSES, HOTELS, AND SCHOOLS.

HEIGHT OF WALLS. Measured from the Curb opposite Center of Building.	FOUNDATIONS.		OUTSIDE AND BEARING WALLS.	REMARKS.
	Stone.	Brick.		
Not exceeding 35 feet.	20″	16″	12″ in basement, 8″ above.	8″ partition walls may be built not exceeding 50 ft. in height.
35 feet and not over 50 feet.	20″	16″	12″ above foundation.	
50 feet and not over 60 feet.	24″	20″	16″ in first story if level with ground, 12″ above.	The height in all cases to be taken to the nearest tier of floor beams.
60 feet and not over 75 feet.	24″	20″	16″ for 25 ft. and 12″ above.	
75 feet and not over 85 feet.	28″	24″	20″ for 20 ft., 16″ to 60 ft., 12″ above.	
85 feet and not over 100 feet.	32″	28″	24″ for 35 ft., 20″ to 75 ft., 16″ above.	Non-bearing walls may be 4″ less in thickness, but not less than 12″.
100 feet and not over 115 feet.	36″	32″	28″ for 25 ft., 24″ to 50 ft., 20″ to 90 ft., 16″ above.	

Walls exceeding 115 feet in height to be increased at the bottom 4″ for every additional 25 ft. in height or part thereof, the upper 115 feet remaining the same as specified for walls of that height.

WALLS FOR WAREHOUSES.

Not exceeding 40 feet.	20″	16″	12 inches.	If there is to be a clear span of over 25 ft. between walls, the bearing walls shall be 4″ more in thickness than here specified for every 12½ ft. or fraction thereof, that said walls are more than 25 ft. apart.
40 feet and not over 60 feet.	24″	20″	16″ for 40 ft., 12″ above.	
60 feet and not over 75 feet.	28″	24″	20″ for 25 ft., 16″ above.	
75 feet and not over 85 feet.	32″	28″	24″ for 25 ft., 20″ to 60 ft., 16″ above.	
85 feet and not over 100 feet.	36″	32″	28″ for 25 ft., 24″ to 50 ft., 20″ to 75 ft., 16″ above.	

Walls exceeding 100 feet in height to be increased at the bottom 4″ for every additional 25 feet in height or part thereof, the upper 100 feet remaining the same as specified for walls of that height.

CHAPTER IV.

COMPOSITION AND RESOLUTION OF FORCES.—CENTRE OF GRAVITY.

LET us imagine a round ball placed on a plane surface at A (Fig. 1), the surface being perfectly level, so that the ball will have no tendency to move until some force is imparted to it. If, now, we impart a force, P, to the ball in the direction indicated by the arrow, the ball will move off in the same direction. If, instead of imparting only one force, we impart two forces, P and P_1, to the ball, it will not move in the direction of either of the forces, but will move off in the direction of the resultant of these forces, or in the direction Ab in the figure. If the magnitude of the forces P and P_1 is indicated by the length of the arrows, then, if we complete the parallelogram $ABCD$, the diagonal DA will represent the direction and magnitude of a force which will have the same effect on the ball as the two forces P_1 and P. If, in addition to the two forces P_1 and P, we now apply a third force, P_2, the ball will move in the direction of the resultant of all three forces, which can be obtained by completing the parallelogram $ADEF$, formed by the resultant DA and the third force P_2. The diagonal R of this second parallelogram will be the resultant of all three of the forces, and the ball will move in the direction Ae. In the same way we could find the resultant of any number of forces.

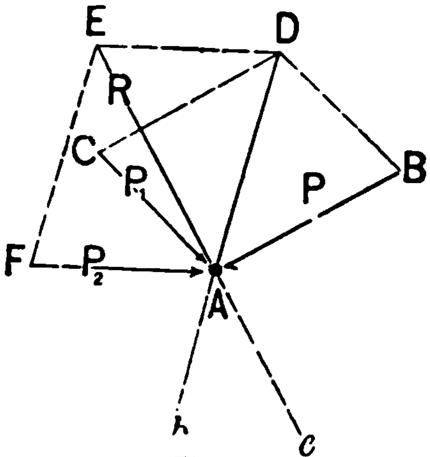


Fig. 1.

Again: suppose we have a ball suspended in the air, whose weight is indicated by the line W (Fig. 2). Now, we do not wish to suspend this ball by a vertical line above it, but by two inclined lines or forces, P and P_1. What shall be the magnitude of these two forces to keep the ball suspended in just this position? We have here just the opposite of our last case; and, instead of finding the diagonal of the resultant, we have the diagonal, which is the line W, and wish to find the sides of the parallelogram. To do this, prolong P and P_1, and from B draw lines parallel to these

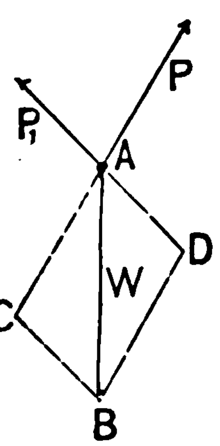


Fig. 2.

to complete the parallelogram. Then will CA be the required magnitude for P, and CB for P_1.

Thus we see how one force can be made to have the same effect as many, or manv can be made to do the work of one. Bearing the above in mind, we are now prepared to study the following propositions:—

I. *A force may be represented by a straight line.*

In considering the action of forces, either in relation to structures or by themselves, it is very convenient to represent the force graphically, which can easily be done by a straight line having an arrow-head, as in Fig. 3. The length of the line, if drawn to a scale of pounds, shows the value of the force in pounds; the direction of the line indicates the direction of the force; the arrow-head shows which way it acts; and the point A denotes the point of application. Thus we have the direction, magnitude, and point of application of the force represented, which is all that we need to know.

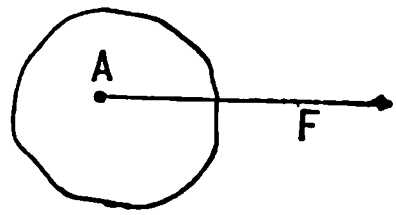


Fig. 3.

Parallelogram of Forces.—II. *If two forces applied at one point, and acting in the same plane, be represented by two straight lines inclined to each other, their resultant will be equal to the diagonal of the parallelogram formed on these lines.*

Thus, if the lines AB and AC (Fig. 4) represent two forces acting on one point, A, and in the same plane, then, to obtain the force which would have the same effect as the two forces, we complete the parallelogram $ABDC$, and draw the diagonal AD. This line will then represent the resultant of the two forces.

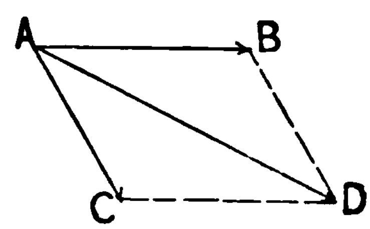


Fig. 4.

When the two given forces are at right angles to each other, the resultant will, by geometry, be equal to the square root of the sum of the squares of the other two forces.

The Triangle of Forces.—III. *If three forces acting on a point be represented in magnitude and direction by the sides of a triangle taken in order, they will keep the point in equilibrium.*

Thus let P, Q, and R (Fig. 5) represent three forces acting on the point O. Now, if we can draw a triangle like that shown at the right of Fig. 5, whose sides shall be respectively parallel to the forces, and shall have the same relation to each other as do the forces, then the

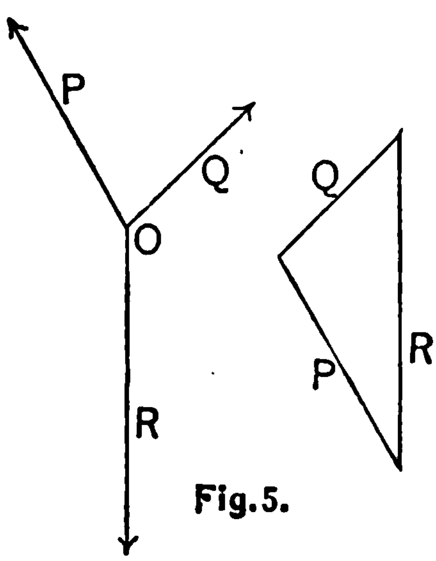


Fig.5.

forces will keep the point in equilibrium. If such a triangle cannot be drawn, the forces will be unbalanced, and the point will not be in equilibrium.

The Polygon of Forces.—IV. *If any number of forces acting at a point can be represented in magnitude and direction by the sides of a polygon taken in order, they will be in equilibrium.*

This proposition is only the preceding one carried to a greater extent.

Moments.—In considering the stability of structures and the strength of materials, we are often obliged to take into consideration the *moments* of the forces acting on the structure or piece; and a knowledge of what a moment is, and the properties of moments, is essential to the proper understanding of these subjects.

When we speak of the moment of a force, we must have in mind some fixed point about which the moment is taken.

The moment of a force about any given point may be defined as the product of the force into the perpendicular distance from the point to the line of action of the force; or, in other words, the moment of a force is the *product of the force by the arm with which it acts.*

Thus if we have a force F (Fig. 6), and wish to determine its moment about a point P, we determine the perpendicular distance Pa, between the point and the line of action of the force, and multiply it by the force in pounds. For example, if the force F were equal to a weight of 500 pounds, and the distance Pa were 2 inches, then the moment of the force about the point P would be 1000 inch-pounds.

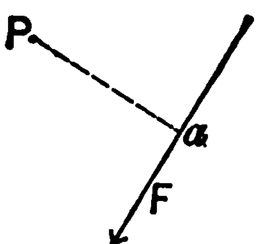


Fig. 6

The following important propositions relating to forces and moments should be borne in mind in calculating the strength or stability of structures.

V.—*If any number of parallel forces act on a body, that the body shall be in equilibrium, the sum of the forces acting in one direction must equal the sum of the forces acting in the opposite direction.*

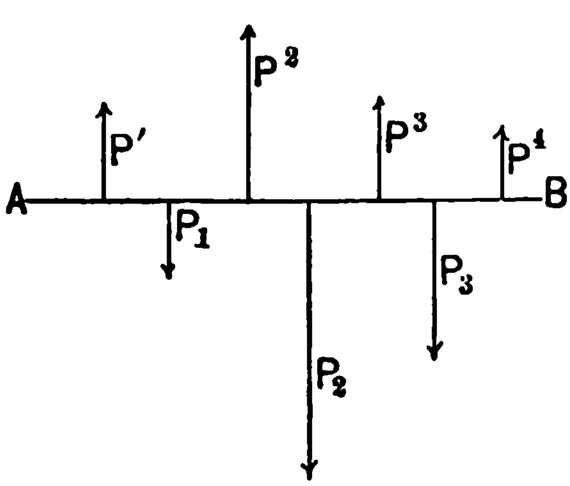


Fig. 7

Thus if we have the parallel forces P^1, P^2, P^3, and P^4, acting on the rod AB (Fig. 7), in the opposite direction to the forces P_1, P_2, P_3, then, if the rod is in equilibrium, the sum of the forces P^1, P^2, P^3, and P^4, must equal the sum of the forces P_1, P_2, and P_3.

VI. *If any number of parallel forces act on a body in opposite directions, then, for the body to be in equilibrium, the sum of the moments tending to turn the body in one direction must equal the sum of the moments tending to turn the body in the opposite direction about any given point.*

Thus let Fig. 8 represent three parallel forces acting on a rod AB. Then, for the rod to be in equilibrium, the sum of the forces F_2 and F_3 must be equal to F_1. Also, if we take the end of the rod, A, for our axis, then must the moment of F_1 be equal to the sum of the moments of F_2 and F_3 about that point, because the moment of F_1 tends to turn the rod down to the right, and the moments of F_2 and F_3 tend to turn the rod up to the left, and there should be no more tendency to turn the rod one way than the other. For example, let the forces F_3, F_2, each be represented by 5, and let the distance Aa be represented by 2, and the distance Ac by 4. The force F_1 must equal the sum of the forces F_3 and F_2, or 10; and its moment must equal the sum of the moments of F_3 and F_2. If we take the moments around A, then the moment of $F_3 = 5 \times 2 = 10$, and of $F_2 = 5 \times 4 = 20$. Their sum equals 30: hence the moment of F_1 must be 30. Dividing the moment 30 by the force 10, we have for the arm 3; or the force F_1 must act at a distance 3 from A to keep the rod in equilibrium.

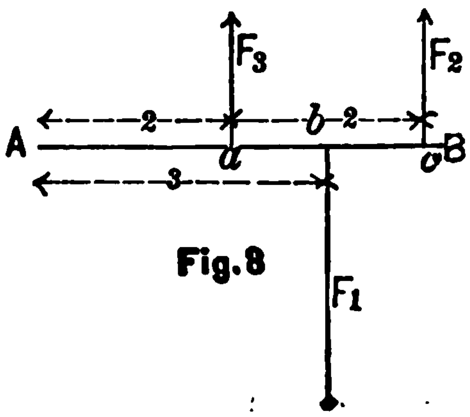


Fig. 8

If we took our moments around b, then the force F_1 would have no moment, not having any arm, and so the moment of F_2 about b must equal the moment of F_3 about the same point; or, as in this case the forces are equal, they must both be applied at the same distance from b, showing that b must be halfway between a and c, as was proved before.

The Principle of the Lever.—This principle is based upon the two preceding propositions, and is of great importance and convenience.

VII. *If three parallel forces acting in one place balance each other, then each force must be proportional to the distance between the other two.*

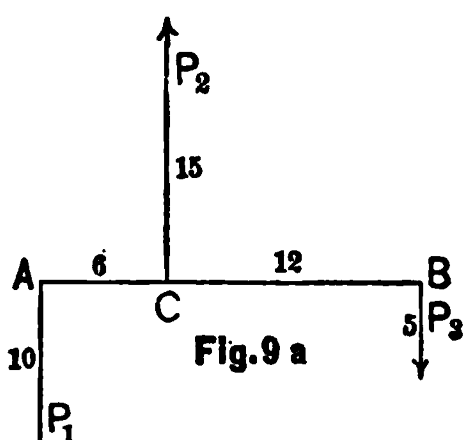


Fig. 9 a

Thus, if we have a rod AB (Figs. 9a, 9b, and 9c), with three forces, P_1, P_2, P_3, acting on it, that the rod shall be balanced, we must have the

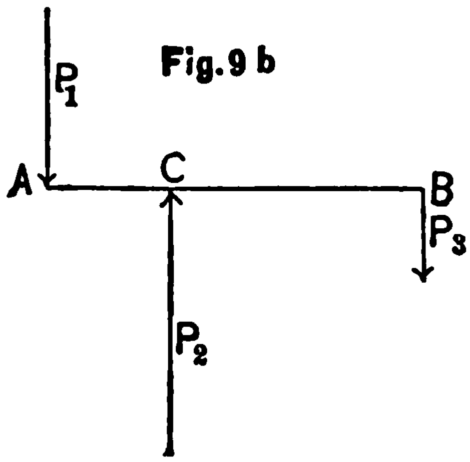
following relation between the forces and their points of application; viz., —

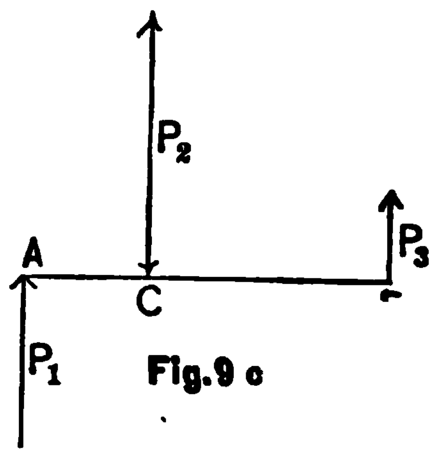
$$\frac{P_1}{CB} : \frac{P_2}{AB} : \frac{P_3}{AC};$$

or

$$P_1 : P_2 : P_3 :: BC : AB : AC.$$

This is the case of the common lever, and gives the means of determining how much a given lever will raise.





The proportion is also true for any arrangement of the forces (as shown in Figs. a, b, and c), provided, of course, the forces are lettered in the order shown in the figures.

EXAMPLE. — Let the distance AC be 6 inches, and the distance CB be 12 inches. If a weight of 500 pounds is applied at the point B, how much will it raise at the other end, and what support will be required at C (Fig. 9b)?

Ans. Applying the rule just given, we have the proportion: —

$$P_3 : P_1 :: AC : CB, \text{ or } 500 : (P_1) :: 6 : 12.$$

Hence $P_1 = 1000$ pounds; or 500 pounds applied at B will lift 1000 suspended at A. The supporting force at C must, by proposition V., be equal to the sum of the forces P_1 and P_3, or 1500 pounds in this case.

Centre of Gravity. — The lines of action of the force of gravity converge towards the centre of the earth; but the distance of the centre of the earth from the bodies which we have occasion to consider, compared with the size of those bodies, is so great, that we may consider the lines of action of the forces as parallel. The number of the forces of gravity acting upon a body may be considered as equal to the number of particles composing the body.

The centre of gravity of a body may be defined as the point through which the resultant of the parallel forces of gravity, acting upon the body, passes in every position of the body.

If a body be supported at its centre of gravity, and be turned about that point, it will remain in equilibrium in all positions. The resultant of the parallel forces of gravity acting upon a body is obviously equal to the weight of the body, and if an equal force be applied, acting in a line passing through the centre of gravity of the body, the body will be in equilibrium.

Examples of Centres of Gravity.—*Centre of Gravity of Lines. Straight Lines.*—By a line is here meant a material line whose transverse section is very small, such as a very fine wire.

The centre of gravity of a uniform straight line is at its middle point. This proposition is too evident to require demonstration.

The centre of gravity of the perimeter of a triangle is at the centre of the circle inscribed in the lines joining the centres of the sides of the given triangle.

Thus, let ABC (Fig. 10) be the given triangle. To find the centre of gravity of its perimeter, find the middle points, D, E, and F, and connect them by straight lines. The centre of the circle inscribed in the triangle formed by these lines will be the centre of gravity sought.

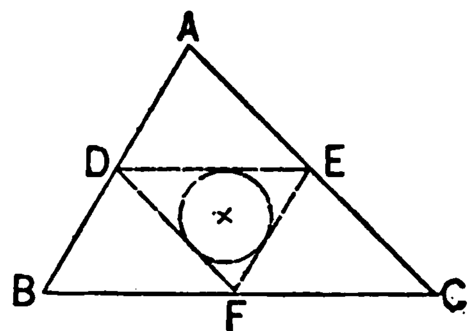


Fig. 10

Symmetrical Lines.—The centre of gravity of lines which are symmetrical with reference to a point will be at that point. Thus the centre of gravity of the circumference of a circle or an ellipse is at the geometrical centre of those figures.

The centre of gravity of the perimeter of an equilateral triangle, or of a regular polygon, is at the centre of the inscribed circle.

The centre of gravity of the perimeter of a square, rectangle, or parallelogram, is at the intersection of the diagonals of those figures.

Centre of Gravity of Surfaces. Definition.—A surface here means a very thin plate or shell.

Symmetrical Surfaces.—If a surface can be divided into two symmetrical halves by a line, the centre of gravity will be on that line: if it can be divided by two lines, the centre of gravity will be at their intersection.

The centre of gravity of the surface of a circle or an ellipse is at the geometrical centre of the figure; of an equilateral triangle or a regular polygon, it is at the centre of the inscribed circle; of a parallelogram, at the intersection of the diagonals; of the surface of a sphere, or an ellipsoid of revolution, at the geometrical centre of the body; of the convex surface of a right cylinder at the middle point of the axis of the cylinder.

Irregular Figures.—Any figure may be divided into rectangles

and triangles, and, the centre of gravity of each being found, the centre of gravity of the whole may be determined by treating the centres of gravity of the separate parts as particles whose weights are proportional to the areas of the parts they represent.

Triangle. — To find the centre of gravity of a triangle, draw a line from each of two angles to the middle of the side opposite: the intersection of the two lines will give the centre of gravity.

Quadrilateral. — To find the centre of gravity of any quadrilateral, draw diagonals, and, from the end of each farthest from their intersection, lay off, toward the intersection, its shorter segment: the two points thus formed with the point of intersection will form a triangle whose centre of gravity is that of the quadrilateral.

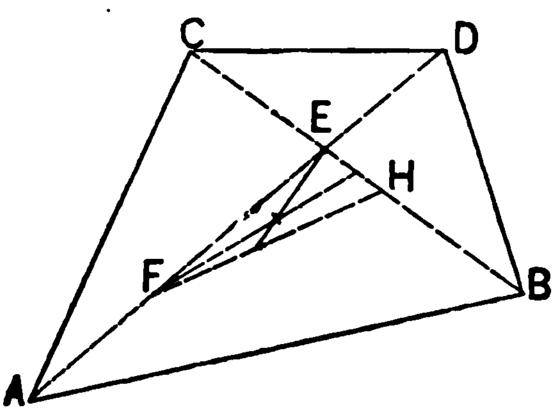


Fig. 11

Thus, let Fig. 11 be a quadrilateral whose centre of gravity is sought. Draw the diagonals AD and BC, and from A lay off $AF = ED$, and from B lay off $BH = EC$. From E draw a line to the middle of FH, and from F a line to the middle of EH. The point of intersection of these two lines is the centre of gravity of the quadrilateral. This is a method commonly used for finding the centre of gravity of the voussoirs of an arch.

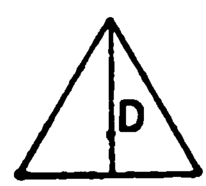
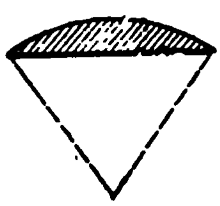
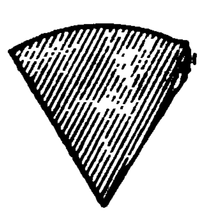


Table of Centres of Gravity. — Let a denote a line drawn from the vertex of a figure to the middle point of the base, and D the distance from the vertex to the centre of gravity. Then


Segment.


Sector.

In an isosceles triangle . . . . . . $D = \frac{2}{3}a$

In a segment of a circle . . . . . $D = \frac{\text{chord}^3}{12 \times \text{area}}$

In a sector of a circle, the vertex being at the centre . $D = R \times \frac{2 \times \text{chord}}{3 \times \text{arc}}$

In a semicircle, vertex being at the centre . . . $D = 0.425R$

In a quadrant of a circle . . . . . . $D = \frac{3}{5}R$

In a semi-ellipse, vertex being at the centre . . . $D = 0.425a$

In a parabola, vertex at intersection of axis with curve) . . . $D = \frac{3}{5}a$

In a cone or pyramid . . . . $D = \frac{3}{4}a$

In a frustum of a cone or pyramid, let h = height of complete cone or pyramid, h' = height of frustum, and the vertex be at apex of complete cone or pyramid; then $D = \frac{3(h^4 - h'^4)}{4(h^3 - h'^3)}$.

The common centre of gravity of two figures or bodies external to each other is found by the following rule:—

Multiply the smaller area or weight by the distance between centres of gravity, and divide the product by the sum of the areas or weights: the quotient will be the distance of the common centre of gravity from the centre of gravity of the larger area.

EXAMPLE.—As an example under the above rule and tables, let us find the common centre of gravity of the semicircle and triangle shown in Fig. 12.

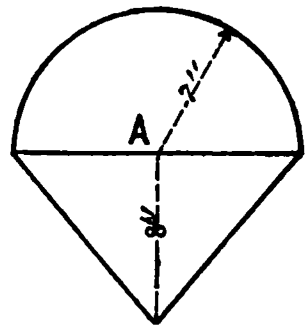


Fig. 12

We must first find the centres of gravity of the two parts.

The centre of gravity of the semicircle is 0.425 R from A, or 2.975. The centre of gravity of the triangle is $\frac{1}{3}$ of 8", or 2.666" from A; and hence the distance between the centre of gravity is 2.975" + 2.666", or 5.641".

The area of the semicircle is approximately $\frac{3\frac{1}{7} \times 49}{2}$, or 77 square inches. The area of the triangle is 7×8, or 56 square inches.

The sum of the areas is 133 square inches. Then, by the above rule, the distance of the common centre of gravity from the centre of gravity of the semicircle is $\frac{56 \times 5.641}{133} = 2.37''$,

or

$$2.975 - 2.37 = 0.605 \text{ inches from } A.$$

Centre of Gravity of Heavy Particles.—*Centre of Gravity of Two Particles.*—Let P be the weight of a particle at A (Fig. 13), and W that at C.

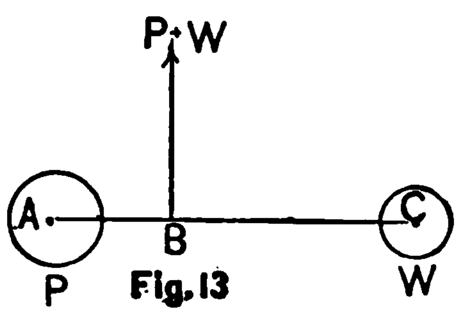


Fig. 13

The centre of gravity will be at some point, B, on the line joining A and C. The point B must be so situated, that if the two particles were held together by a stiff wire, and were supported at B by a force equal to the sum of P and W, the two particles would be in equilibrium.

The problem then comes under the principle of the lever, and hence we must have the proportion,

$$P + W : P :: AC : BC,$$

or

$$BC = \frac{P \times AC}{P + W}.$$

If $W = P$, then $BC = AB$, or the centre of gravity will be half-

way between the two particles. This problem is of great importance, for it presents itself in many practical examples.

Centre of Gravity of Several Heavy Particles. — Let W_1, W_2, W_3, W_4 and W_5 (Fig. 14) be the weights of the particles.

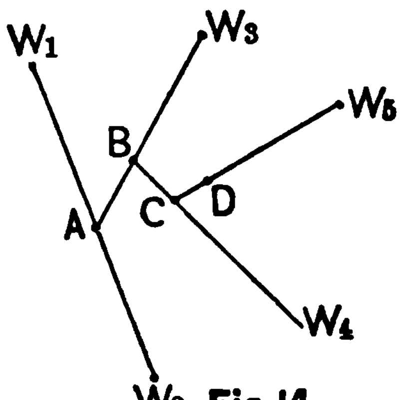


Fig. 14

Join W_1 and W_2 by a straight line, and find their centre of gravity A, as in the preceding problem. Join A with W_3, and find the centre of gravity B, which will be the centre of gravity of the three weights W_1, W_2, and W_3. Proceed in the same way with each weight, and the last centre of gravity found will be the centre of gravity of all the particles.

In both of these cases the lines joining the particles are supposed to be horizontal lines, or else the horizontal projection of the real straight line which would join the points.

CHAPTER V.

RETAINING WALLS.

A Retaining Wall is a wall for sustaining a pressure of earth, sand, or other filling or backing deposited behind it after it is built, in distinction to a *brest* or *face* wall, which is a similar structure for preventing thé fall of earth which is in its undisturbed natural position, but in which a vertical or inclined face has been excavated.

Fig. 1 gives an illustration of the two kinds of wall.

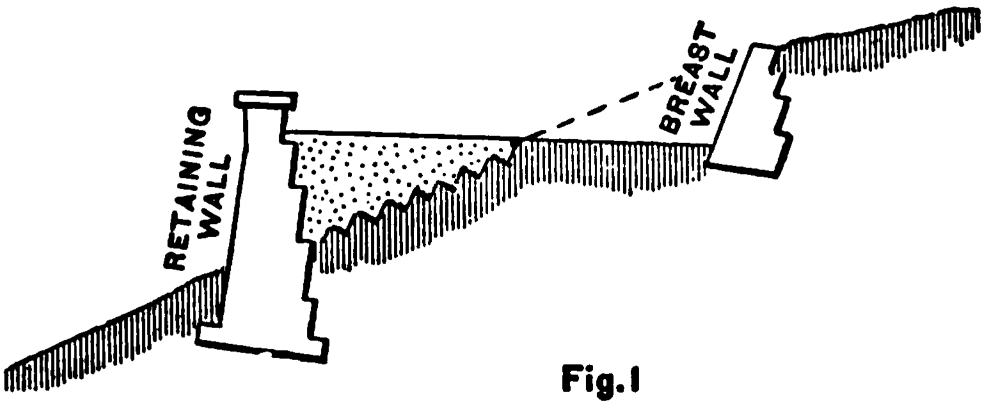


Fig. 1

Retaining Walls. — A great deal has been written upon the theory of retaining walls, and many theories have been given for computing the thrust which a bank of earth exerts against a retaining wall, and for determining the form of wall which affords the greatest resistance with the least amount of material.

There are so many conditions, however, upon which the thrust exerted by the backing depends, — such as the cohesion of the earth, the dryness of the material, the mode of backing up the wall, etc., — that in practice it is impossible to determine the exact thrust which will be exerted against a wall of a given height.

It is therefore necessary, in designing retaining walls, to be guided by experience rather than by theory. As the theory of retaining walls is so vague and unsatisfactory, we shall not offer any in this article, but rather give such rules and cautions as have been established by practice and experience.

In designing a retaining wall there are two things to be considered, — the backing and the wall.

The tendency of the backing to slip is very much less when it is

in a dry state than when it is filled with water, and hence every precaution should be taken to secure good drainage. Besides surface drainage, there should be openings left in the wall for the water which may accumulate behind it to escape and run off.

The manner in which the material is filled against the wall also affects the stability of the backings. If the ground be made irregular, as in Fig. 1, and the earth well rammed in layers inclined *from* the wall, the pressure will be very trifling, provided that attention be paid to drainage. If, on the other hand, the earth be tipped, in the usual manner, in layers sloping *towards* the wall, the full pressure of the earth will be exerted against it, and it must be made of corresponding strength.

Fig.2

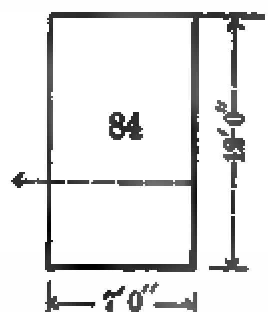


Fig.3

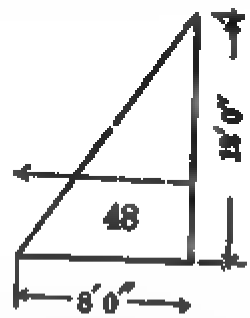


Fig.4

The Wall. — Retaining walls are generally built with a battering (sloping) face, as this is the strongest wall for a given amount of material; and, if the courses are inclined towards the back, the tendency to slide on each other will be overcome, and it will not be necessary to depend upon the adhesion of the mortar.

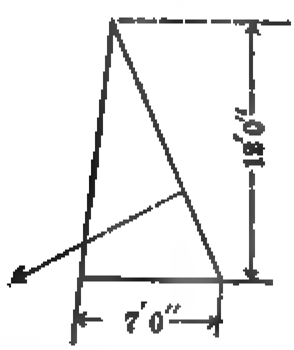


Fig.5

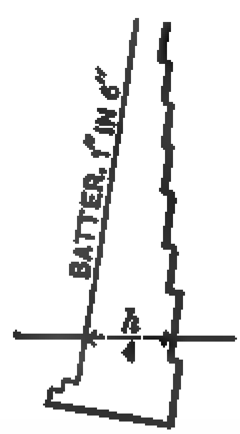


Fig.6

The importance of making the resistance independent of the adhesion of the mortar is obviously very great; as it would otherwise be necessary to delay backing up a wall until the mortar was thoroughly set, which might require several months.

e Back of the Wall should be left Rough.—In work it would be well to let every third or fourth course t an inch or two. This increases the friction of the earth st the back, and thus causes the resultant of the forces acting d the wall to become more nearly vertical, and to fall farther n the base, giving increased stability. It also conduces to gth not to make each course of uniform height throughout the ness of the wall, but to have some of the stones, especially near ck, sufficiently high to reach up through two or three courses. is means the whole masonry becomes more effectually inter-l or bonded together as one mass, and less liable to bulge.

ere deep freezing occurs, the back of the wall should be sloped rds for three or four feet below its top, as at *OC* (Fig. 2), which d be quite smooth, so as to lessen the hold of the frost, and nt displacement.

s. 3, 4, 5, and 6 show the relative sectional areas of walls of ent shapes that would be required to resist the pressure of a of earth twelve feet high ("Art of Building," E. Dobson, . The first three examples are calculated to resist the maxi-thrust of wet earth, while the last shows the modified form ly adopted in practice.

les for the Thickness of the Wall.—As has been , the only practical rules for retaining walls which we have npirical rules based upon experience and practice.

John C. Trautwine, C.E., who is considered authority on eering subjects, gives the following table in his "Pocket-Book ngineers," for the thickness at the base of vertical retaining with a sand-backing deposited in the usual manner.

first column contains the vertical height *CD* (Fig. 7) of the as compared with the vertical height of the wall; which latter

is assumed to be 1, so that the table begins with backing of the same height as the wall. These vertical walls may be battered to any extent not exceeding an inch and a half to a foot, or 1 in 8, without affecting their stability, and without increasing the base.

Proportion of Retaining Walls.

Total height of the earth compared with the height of the wall above ground.	Wall of cut stone in mortar.	Good mortar, rubble, or brick.	Wall of good, dry rubble.
1	0.35	0.40	0.50
1.1	0.42	0.47	0.57
1.2	0.46	0.51	0.61
1.3	0.49	0.54	0.64
1.4	0.51	0.56	0.66
1.5	0.52	0.57	0.67
1.6	0.54	0.59	0.69
1.7	0.55	0.60	0.70
1.8	0.56	0.61	0.71
2	0.58	0.63	0.73
2.5	0.60	0.65	0.75
3	0.62	0.67	0.77
4	0.63	0.68	0.78
6	0.64	0.69	0.79

Brest Walls (from Dobson's "Art of Building"). — Where the ground to be supported is firm, and the strata are horizontal, the office of a brest wall is more to protect than to sustain the earth. It should be borne in mind that a trifling force skilfully applied to unbroken ground will keep in its place a mass of material, which, if once allowed to move, would crush a heavy wall; and therefore great care should be taken not to expose the newly opened ground to the influence of air and wet for a moment longer than is requisite for sound work, and to avoid leaving the smallest space for motion between the back of the wall and the ground.

The strength of a brest wall must be proportionately increased when the strata to be supported inclines towards the wall: where they incline from it, the wall need be little more than a thin facing to protect the ground from disintegration.

The preservation of the natural drainage is one of the most important points to be attended to in the erection of brest walls, as upon this their stability in a great measure depends. No rule can be given for the best manner of doing this: it must be a matter for attentive consideration in each particular case.

CHAPTER VI.

STRENGTH OF MASONRY.

By the term "strength of masonry" we mean its resistance to a crushing-force, as that is the only force to which masonry should be subjected. The strength of the different stones and materials used in masonry, as determined by experiment, is given in the following table. (For Architectural Terra-Cotta, see page 186*a*.)

Crushing Resistance of Brick, Stone, and Concretes. (*Pressure at right angles to bed.*)

	Pounds per sq. inch.
Brick: Common, Massachusetts	10,000
Common, St. Louis	6,417
Common, Washington, D. C	7,370
Paving, Illinois	6,000 to 13,000
Granites: Blue, Fox Island, Me	14,875
Gray, Vinal Haven, Me	13,000 to 18,000
Westerly, R. I	15,000
Rockport and Quincy, Mass	17,750
Milford, Conn	22,600
Staten Island, N. Y	22,250
East St. Cloud, Minn	28,000
Gunnison, Colo	13,000
Red, Platte Cañon, Colo	14,600
Limestones: Glens Falls, N. Y	11,475
Joliet, Ill	12,775
Bedford, Ind	6,000 to 10,000
Salem, Ind	8,625
Red Wing, Minn	23,000
Stillwater, Minn	10,750
Sandstones: Dorchester, N. B. (brown)	9,150
Mary's Point, N. B. (fine grain, dark brown)	7,700
Connecticut Brown Stone,* on bed	7,000 to 13,000
Longmeadow, Mass. (reddish brown)	7,000 to 14,000
" " average, for good quality	12,000
Little Falls, N. Y	9,850
Medina, N. Y	17,000
Potsdam, N. Y. (red)	18,000 to 42,000
Cleveland, Ohio	6,800
North Amherst, Ohio	6,212
Berea, Ohio	8,000 to 10,000
Hummelstown, Pa	12,810
Fond du Lac, Minn	8,750
Fond du Lac, Wis	6,237
Manitou, Colo. (light red)	6,000 to 11,000
St. Vrain, Colo. (hard laminated)	11,505
Marbles: Lee, Mass	22,900
Rutland, Vt	10,746
Montgomery Co., Pa	10,000
Colton, Cal	17,783
Italy	12,156
Flagging: North River, N. Y	13,425
Concrete: Rosendale cement 1, sand and stone 7½, 46 months old	1,544
Portland cement 1, sand and stone 9, 6 months	2,000

* This stone should not be set *on edge*.

The stones in this table are supposed to be *on bed*, and the height to be not more than four times the least side. Of the strength of rubble masonry, Professor Rankine states that "the resistance of *good coursed rubble* masonry to crushing is about four-tenths of that of single blocks of the stone it is built with. The resistance of *common rubble* to crushing is not much greater than that of the mortar which it contains."

Stones generally commence to crack or split under about one-half of their crushing-load.

Crushing-Height of Brick and Stone. — If we assume the weight of brickwork to be 112 pounds per cubic foot, and that it would crush under 450 pounds per square inch, then a vertical uniform column 580 feet high would crush at its base under its own weight.

Average sandstones at 145 pounds per cubic foot would require a column 5950 feet high to crush it; and average granite at 165 pounds per cubic foot would require a column 10,470 feet high. The Merchants' shot-tower at Baltimore is 246 feet high, and its base sustains a pressure of six tons and a half (of 2240 pounds) per square foot. The base of the granite pier of Saltash Bridge (by Brunel) of solid masonry to the height of 96 feet, and supporting the ends of two iron spans of 455 feet each, sustains nine tons and a half per square foot. The highest pier of Rocquefavour stone aqueduct, Marseilles, is 305 feet, and sustains a pressure at the base of thirteen tons and a half per square foot.

Working-Strength of Masonry. — The working-strength of masonry is generally taken at from one-sixth to one-tenth of the crushing-load for piers, columns, etc., and in the case of arches a factor of safety of twenty is often recommended for computing the resistance of the voussoirs to crushing.

Mr. Trautwine states that it cannot be considered safe to expose even first-class pressed brickwork in *cement* to more than thirteen or sixteen tons' pressure per square foot, or good hand-moulded brick to more than two-thirds as much. (*See page* 181.)

Sheet lead is sometimes placed at the joints of stone columns with a view to equalize the pressure, and thus increase the strength of the column. Experiments, however, seem to show that the effect is directly the reverse, and that the column is materially weakened thereby.[1]

Piers. — Masonry that is so heavily loaded that it is necessary to proportion it in regard to its strength to resist crushing, is, as a general rule, in the form of piers, either of brick or stone. As

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[1] Trautwine's Pocket-book, p. 176.

these piers are often in places where it is desirable that they should occupy as little space as possible, they are often loaded to the full limit of safety.

The material generally used for building piers is brick: block or cut stone is sometimes used, for the sake of appearance; but rubble-work should never be used for piers which are to sustain posts, pillars, or columns. Brick piers more than six feet in height should never be less than twelve inches square, and should have properly proportioned footing courses of stone not less than a foot thick.

The brick in piers should be hard and well burned, and should be laid in cement, or cement mortar at least, and be well wet before being laid, as the strength of a pier depends very much upon the mortar or cement with which it is laid: those piers which have to sustain very heavy loads should be built up with the best of the Rosendale cements. The size of the pier should be determined by calculating the greatest load which it may ever have to sustain, and dividing the load by the safe resistance of one square inch or foot of that kind of masonry to crushing.

Example. — In a large storehouse the floors are supported by a girder running lengthwise through the centre of the building. The girders are supported every twelve feet by columns, and the lowest row of columns is supported on brick piers in the basement. The load which may possibly come upon one pier is found to be 65,000 pounds. What should be the size of the pier?

Ans. The masonry being of good quality, and laid in cement mortar, we will assume 12 tons per square foot, or 166 lbs. per square inch (see p. 181), for the safe working load. Dividing 65,000 lbs. by 166, we have 391 square inches for the size of the pier. This would require a pier 20 × 20 inches.

It is the custom with many architects to specify bond stones in brick piers (the full size of the section of the pier) every three or four feet in the height of the pier. These bond stones are generally about four inches thick. The object in using them is to distribute the pressure on the pier equally through the whole mass. Many first-class builders, however, consider that the piers are stronger without the bond stone; and it is the opinion of the writer that a pier will be just as strong if they are not used.

Section 3 of the Building Laws of the city of New York requires that every isolated pier less than ten superficial feet at the base, and all piers supporting a wall built of rubble-stone or brick, or under any iron beam or arch-girder, or arch on which a wall rests, or lintel supporting a wall, shall, at intervals of not less than thirty inches in height, have built into it a bond stone not less than

four inches thick, of a diameter each way equal to the diameter of the pier, except that in piers on the street front, above the curb, the bond stone may be four inches less than the pier in diameter.

Piers which support columns, posts, or pillars, should have the top covered by a plate of stone or iron, to distribute the pressure over the whole cross-section of the pier.

In Boston, it is required that "all piers shall be built of good, hard, well-burned brick, and laid in clear cement, and all bricks used in piers shall be of the hardest quality, and be well wet when laid.

"Isolated brick piers under all lintels, girders, iron or other columns, shall have a cap-iron at least two inches thick, or a granite cap-stone at least twelve inches thick, the full size of the pier.

"Piers or columns supporting walls of masonry shall have for a footing course a broad leveller, or levellers, of block stone not less than sixteen inches thick, and with a bearing surface equal in area to the square of the width of the footing course plus one foot required for a wall of the same thickness and extent as that borne by the pier or column."

For the *Strength of Masonry Walls*, see Chap. III.

The following tables give the results of some **tests on brick, brick piers, and stone,** made under the direction of the author, in behalf of the Massachusetts Charitable Mechanics Association.

The specimens were tested in the government testing-machine at Watertown, Mass., and great care was exercised to make the tests as perfect as possible. As the parallel plates between which the brick and stone were crushed are fixed in one position, it is necessary that the specimen tested should have perfectly parallel faces.

The bricks which were tested were rubbed on a revolving bed until the top and bottom faces were perfectly true and parallel.

The preparation of the bricks in this way required a great deal of time and expense; and it was so difficult to prepare some of the harder brick, that they had to be broken, and only one-half of the brick prepared at a time.

TABLE

Showing the Ultimate and Cracking Strength of the Brick, the Size and Area of Face.

NAME OF BRICK.	Size.	Area of face in sq. ins.	Commenced to crack under lbs. per sq. inch.	Net strength lbs. per sq. inch.
Philadelphia Face Brick . . .	Whole brick	33.7	4,303	6,062
" " " . . .	Whole brick	32.2	3,400	5,831
" " " . . .	Whole brick	34.03	2,879	5,862
Average	. . . . . .	. . .	**3,527**	**5,918**
Cambridge Brick (Eastern) .	Half brick .	10.89	3,670	9,825
" " " .	Whole brick	25.77	7,760	12,941
" " " .	Half brick .	12.67	3,393	11,681
" " " .	Half brick .	13.43	3,797	14,296
Average	. . . . . .	. . .	**4,655**	**12,186**
Boston Terra-Cotta Co.'s Brick,	Half brick .	11.46	11,518	13,839
" " " "	Whole brick	25.60	8,593	11,406
" " " "	Whole brick	28.88	3,530	9,766
Average	. . . . . .	. . .	**7,880**	**11,670**
New-England Pressed Brick .	Half brick .	12.95	3,862	10,270
" " " .	Half brick .	13.2	8,180	13,530
" " " .	Half brick .	13.30	2,480	13,082
" " " .	Half brick .	13.45	4,535	13,085
Average	. . . . . .	. . .	**4,764**	**12,490**

The Philadelphia Brick used in these tests were obtained from a Boston dealer, and were fair samples of what is known in Boston as Philadelphia Face Brick. They were a very soft brick.

The Cambridge Brick were the common brick, such as is made around Boston. They are about the same as the Eastern Brick.

The Boston Terra-Cotta Company's Brick were manufactured of a rather fine clay, and were such as are often used for face brick.

The New-England Pressed Brick were hydraulic pressed brick, and were almost as hard as iron.

From tests made on the same machine by the United States Government in 1884, the average strength of three (M. W. Sands) Cambridge, Mass., face brick was 13,925 pounds, and of his common brick, 18,337 pounds per square inch, one brick developing the enormous strength of 22,351 pounds per square inch. This was a very hard-burnt brick.

Three brick of the Bay State (Mass.) manufacture showed an average strength of 11,400 pounds per square inch.

The New England brick are among the hardest and strongest brick in the country, those in many parts of the West not having one-fourth of the strength given above, so that in heavy buildings,

where the strength of the brick to be used is not known by actual tests, it is advisable to have the brick tested.

Prof. Ira O. Baker, of the University of Illinois, reported some tests on Illinois brick, made on the 100,000 pounds testing machine at the university, in 1888–89, which gives the crushing strength of soft brick at 674 pounds per square inch, average of three face brick, 3,070 pounds; and of four paving brick, 9,775 pounds.

In nearly all makes of brick it will be found that the face brick are not as strong as the common brick.

Tests of the Strength of Brick Piers laid with Various Mortars.[1]—These tests were made for the purpose of testing the strength of brick piers laid up with different cement mortars, as compared with those laid up with ordinary mortar. The brick used in the piers were procured at M. W. Sands's brick-yard, Cambridge, Mass., and were good ordinary brick. They were from the same lot as the samples of common brick tested as described.

The piers were 8″ by 12″, and nine courses, or about 22½″ high, excepting the first, which was but eight courses high. They were built Nov. 29, 1881, in one of the storehouses at the United-States Arsenal in Watertown, Mass. In order to have the two ends of the piers perfectly parallel surfaces, a coat of about half an inch thick of pure Portland cement was put on the top of each pier, and the foot was grouted in the same cement.

March 3, 1882, three months and five days later, the tops of the piers were dressed to plane surfaces at right angles to the sides of the piers. On attempting to dress the lower ends of the piers, the cement grout peeled off, and it was necessary to remove it entirely, and put on a layer of cement similar to that on the top of the piers. This was allowed to harden for one month and sixteen days, when the piers were tested. At that time the piers were four months and twenty-six days old. As the piers were built in cold weather, the bricks were not wet.

The piers were built by a skilled brick-layer, and the mortars were mixed under his superintendence. The tests were made with the government testing-machine at the Arsenal.

The following table is arranged so as to show the result of these tests, and to afford a ready means of comparison of the strength of brickwork with different mortars. The piers generally failed by cracking longitudinally, and some of the brick were crushed. The

[1] The report of these tests was first published in the American Architect, June 3, 1882.

Portland cement used in these tests was known as Brooks, Shoobridge & Co.'s cement.

As the actual strength of brick piers is a very important consideration in building construction, the following tests, made by the United States Government at Watertown, Mass., and contained in the report of the tests made on the Government testing machine for the year 1884. are given, as being of much value.

Three kinds of brick were represented in the construction of the piers, and mortars of different composition—ranging in strength from lime mortar to neat Portland cement. The piers ranged in cross-section dimensions from 8" × 8" to 16" × 16", and in height from 16" to 10'.

The piers were tested at the age of from 18 to 24 months

The following table gives the results obtained, and memoranda regarding the size and character of the piers.

TABULATED RESULTS OF THE

ACTUAL CRUSHING STRENGTH OF BRICK PIERS.

BUILT OF FACE-BRICKS (M. W. SANDS, CAMBRIDGE, MASS.).

No. of Test.	Nominal dimensions. Height		Cross-section.		Composition of mortar.			First crack.	Ultimate strength. Total	Per sq. in.	Per sq. ft	% of single brick.
	Ft.	In	In	In				lbs.	lbs.	lbs.	Tons.	
11	1	4	8	8	1 lime, 3 sand,				148,600	2,52)	181 4	18.1
330	6	8	8	8	" "				108,400	1,677	135.1	13 5
12	1	4	8	8	1 Portland cement, 2 sand,				218,100	3,7.6	271.8	27.1
331	6	8	8	8	" "				129,900	2,240	161.9	16 2
333	2	0	12	12	1 lime, 3 sand,				257,100	1,940	139.7	13 9
334[1]	2	0	12	12	" "				226,100	1,990	143 3	14 3
332	10	0	12	12	" "				199,800	1,511	108.8	10 9
334[2]	10	0	12	12	" "				208,600	1,807	130.1	13.0
335	2	0	12	12	1 Portland cement, 2 sand,				488,000	3,670	264.2	26.4
336	10	0	12	12	" "				298,000	2,253	162 2	16.2

BUILT OF COMMON BRICKS (M. W. SANDS).

No. of Test.	Height Ft.	In	Cross-section In	In	Composition of mortar.			First crack.	Total	Per sq. in.	Per sq. ft	% of single brick.
10	1	4	8	8	1 lime, 3 sand,	125.6	60.80	66,000	148,800		175 6	18.8
13½	6	8	8	8	" "	133.6	62.40	....	96,100		110.8	8.4
251	2	0	12	12	" "	......	138.06	75,000	296,400		154 8	11.7
[illegible][3]	2	0	12	12	" "	......	119.88	190,000	244,600		147 6	11.2
261	[illegible]	[illegible]	12	12	" "	131.5	138 06	70,000	154,300		80.5	6.1
[illegible][4]	10	0	12	12	" "	136.0	115.50	70,000	183,800		114 8	8.6
[illegible]	10	0	12	12	1 Portland cement, 2 sand,	131.0	138.06	....	276.000		144.2	10.9
[illegible]	2	8	16	16	" " "	....	256.00	460.000	696,000	[illegible]	105.8	14.8
[illegible]	10	0	16	16	" " "	....	256.00	340,000	483,100	1,887	135.8	10.3

[1] Has hollow core 4.25" x 4.25". [2] Has hollow core 4.1" x 4.1". [3] Has hollow core 4.5" x 4.5". [4] Has hollow core 4 75" x 4.75".

BUILT OF COMMON BRICKS (BAY STATE).

295	2	0	12	12	1 lime, 3 sand,	....	146.41	1,370	98.6	12.0
298	6	0	12	12	" "		144.00	1,138	81.6	9.9
299	6	0	12	12	" "	119.7	144.00	1,210	87.1	10.6
291[a]	6	0	12	12	" "	118.3	144.00	1,331	95.8	11.7
292[b]	6	0	12	12	" "	116.1	156.25	1,211	87.2	10.6
305	7	10	12	12	" "	120	144.00	1,174	84.6	10.3
297	10	0	12	12	" "	116	144.00	924	66.6	8.1
335	10	0	8	12	" "	107.0	96.00	940	67.7	8.2
303	10	0	12	16	" "	118.7	192.00	773	55.7	6.8
301	6	6	12	12	1 Rosendale cement, 2 lime-mortar,	120.6	144.00	1,646	118.5	14.4
302	6	0	12	12	" " 2 sand,	123.0	144.00	1,972	142.0	17.3
300	6	0	12	12	1 Portland cement, 2 lime-mortar,	120.3	144.00	1,411	101.6	12.4
294	6	0	12	12	" " 2 sand,	119.7	144.00	1,792	129.0	15.7
290	6	0	12	12	Neat Portland cement,	126.6	144.00	2,375	171.0	20.8

[a] Joints broken every 6 courses.

[b] Bricks laid on edge.

Tests of Mortar Cubes.—The following tests of 6″ cubes of mortar were made by the United States Government at Watertown, Mass., in the year 1884.

The mortar cubes were allowed to season in the open air, a period of fourteen and a half months, when they were tested.

The age of the plaster cube was four months. It should be noticed that, while the cubes of Rosendale cement and lime-mortar showed a greater strength than when sand alone was mixed with the cement, with the cubes of Portland cement and lime-mortar the reverse was the case, differing from the result obtained by the author. This shows the necessity of a number and variety of tests.

TABULATED RESULTS, 6″ MORTAR CUBES.

CRUSHING STRENGTH.

No. of test.	Composition.	First crack.	Ultimate strength per sq. in.	Weight per cu. ft.
		lbs.	lbs.	lbs.
3 *a*	1 part lime, 3 parts sand,	....	135	112
3 *b*	" " "	....	119	111
3 *c*	" " "	....	118	106
4 *a*	1 part Portland cement, 2 parts sand,	....	560	116
4 *b*	" " " "	....	696	120
4 *c*	" " " "	11,500	883	115
5 *a*	1 part Rosendale cement, 2 parts sand,	....	156	111
5 *b*	" " " "	....	186	109
5 *c*	" " " "	4,500	143	107
6 *a*	Neat Portland cement,	... .	2,673	126
6 *b*	" "	95,000	3,548	129
6 *c*	" "	....	4,227	135
7 *a*	Neat Rosendale cement,	11,000	421	94
7 *b*	" "	19,000	615	99
7 *c*	" "	19,200	526	97
8 *a*	1 part Portland cement, 2 parts lime-mortar,[1]	....	204	100
8 *b*	" " " "	....	196	110
8 *c*	" " " "	....	175	103
9 *a*	1 part Rosendale cement, 2 parts lime-mortar,[1]	.. ....	194	105
9 *b*	" " " "	....	193	106
9 *c*	" " " "	....	162	105
	Plaster-of-Paris.	....	1,981	74

Working Strength of Masonry.—The following table has been compiled as representing the practice of leading engineers, and the average requirements of recent building laws. The author believes that the values may be relied upon with safety, and with-

[1] Lime-mortar, 1 part lime, 3 parts sand.

out undue waste of materials. For the size of cast-iron bearing plates on masonry, see page 242*b*. For strength of architectural terra-cotta, see page 186*a*.

SAFE WORKING LOADS FOR MASONRY.

Brickwork in walls or piers.

	TONS PER SQUARE FOOT. Eastern.	Western.
Red brick in lime mortar	7	5
" hydraulic lime mortar		6
" natural cement mortar, 1 to 3	10	8
Arch or pressed brick in lime mortar	8	6
" " " natural cement	12	9
" " " Portland cement	15	12½

Piers exceeding in height six times their least dimensions should be increased 4 inches in size for each additional 6 feet.

Stonework.

(Tons per square foot.)

Rubble walls, irregular stones	3
" coursed, soft stone	2½
" " hard stone	5 to 16
Dimension stone, squared in cement:	
Sandstone and limestone	10 to 20
Granite	20 to 40
Dressed stone, with ⅛-inch dressed joints in cement:	
Granite	60
Marble or limestone, best	40
Sandstone	30

Height of columns not to exceed eight times least diameter.

Concrete.

Portland cement, 1 to 8	8 to 15
Rosendale cement, 1 to 6	5 to 10
Hydraulic lime, best, 1 to 6	5

Hollow Tile.

(Safe loads per square inch of effective bearing parts.)

Hard fire-clay tiles	80 lbs.
" ordinary clay tiles	60 "
Porous terra-cotta tiles	40 "

Mortars.

(In ½-inch joints, 3 months old, tons per square foot.)

Portland cement, 1 to 4	40
Rosendale cement, 1 to 3	13
Lime mortar, best	8 to 10
Best Portland cement, 1 to 2, in ¼-inch joints for bedding iron plates	70

Actual Tests of the Crushing-Strength of Sandstones (made under the direction of the author for the Massachusetts Charitable Mechanics' Association).—These tests were made with the Government testing machine at the United States Arsenal, Watertown, Mass., and every precaution was taken to secure accurate results.

WOOD'S POINT (N.B.) SANDSTONE. — This stone is of about the same color as the Mary's Point stone, but it has a much coarser grain, and is not very hard.

Block No. 1 measured 4.03″ × 4.03″ × 8″. Sectional area 16.2 square inches.

Commenced to crack at 50,000 pounds, on the corners, and continued cracking, along the edges and at the corners, until it was crushed under 80,000 lbs.' pressure, or 4932 lbs. per square inch.

Block No. 2 measured 4″ × 3.98″ × 7.25″. Sectional area 15.92 square inches.

This stone commenced to crack under a pressure of 44,000 pounds, and failed under a pressure of 62,500 pounds, or 3976 pounds per square inch.

LONGMEADOW STONE. — The Bay of Fundy Quarrying Company also quarry a variety of the Longmeadow (Mass.) sandstone, which is a reddish-brown in color.

Block No. 1 measured 3.80″ × 3.87″ × 7.42″. Sectional area 14.71 square inches.

This stone showed no cracks whatever until the pressure had reached 152,000 pounds, when it commenced to crack at the corners. When the pressure reached 200,000 pounds, the stone suddenly flew from the machine in fragments, giving an ultimate strength of 13,596 pounds per square inch.

This stone did not fit into the machine very perfectly.

Block No. 2 measured 3.39″ × 3.97″ × 7.5″. Sectional area 15.6 square inches.

The stone commenced to crack along the edges under a pressure of 47,000 pounds. Under 78,000 pounds, a large piece of the stone split off from the bottom of the block, and under 142,300 pounds' pressure, the stone failed, cracking very badly. *Ultimate strength per square inch* 9121 *pounds.*

BROWN SANDSTONE FROM EAST LONGMEADOW, MASS. — Quarried by Norcross Brothers & Taylor of East Longmeadow. This firm works several quarries, the stone differing in the degree of hardness, and a little in color, which is a reddish brown. The different varieties take the name of the quarry from which they come.

SOFT SAULSBURY BROWNSTONE. — This stone is one of the

softest varieties quarried by this firm, although it is about as hard as the ordinary brownstones. The specimens tested were selected by the foreman of the stone-yard without knowing the purpose for which they were to be used, and were rather below the average of this stone in quality.

Block No. 1 measured 4″ × 4″ × 7.58″. Area of cross-section 16 square inches. *Ultimate strength* 141,000 *pounds*, or 8812 *pounds per square inch.*

Stone did not commence to crack until the pressure had reached 130,000 pounds.

Block No. 2 measured 4″ × 4″ × 7.85″. Area of cross-section 16 square inches. *Ultimate strength* 129,000 *pounds*, or 8062 *pounds per square inch.*

There were no cracks in the specimen when it was under 100,000 pounds' pressure.

HARD SAULSBURY BROWNSTONE. — This is one of the hardest and finest of the Longmeadow sandstones.

Block No. 1 measured 4.16″ × 4.16″ × 8″. Sectional area 17.3 square inches. *Ultimate strength* 233,900 *pounds*, or 13,520 *pounds per square inch.*

Stone did not commence to crack until the pressure had reached 220,000 pounds, almost the crushing-strength.

Block No. 2 measured 4.15″ × 4.15″ × 8″. Sectional area 17.2 square inches. *Ultimate strength* 252,000 *pounds*, or 14,650 *pounds per square inch.*

This specimen did not commence to crack until the pressure had reached 240,000 pounds, or 13,953 pounds to the square inch.

The following table is arranged to show the sectional area and strength of each specimen, and the average for each variety of stone. The cracking-strength, so to speak, of the stone, is of considerable importance, for, after a stone has commenced to crack, its permanent strength is probably reached; for, if the load which caused it to crack were allowed to remain on the stone, it would probably in time crush the stone. In testing the blocks, however, some inequality in the faces of the block might cause one corner to crack when the stone itself had not commenced to weaken.

Gen. Q. A. Gillmore, a few years ago, tested the strength many varieties of sandstone by crushing two-inch cubes. The r sults obtained by him ranged from 4350 pounds to 9650 pounds p square inch. Comparing the strength of the stones tested by th author with these values, we find that the specimens of Ha Saulsbury sandstone had a strength far above the average for san stones, and the other specimens have about the same values those obtained by Gen. Gillmore.

We should expect, however, smaller values from blocks 4″ × 4 × 8″ than from two-inch cubes; for, as a rule, small specimens almost any material show a greater strength than large specimens

It is interesting to note the mode of fracture of the blocks brownstone, which was the same for each specimen. The block fractured by the sides bursting off; and, when taken from the

chine, the specimens had the form of two pyramids, with their apexes meeting at the centre, and having for their bases the compressed ends of the block. The pyramids were more clearly shown in some specimens than in others, but it was evident that the mode of fracture was the same for all specimens.

KIBBE SANDSTONE.—In 1883 the writer superintended the testing of two six-inch cubes of the Kibbe variety of Longmeadow sandstone, quarried by Norcross Brothers. One block withstood a pressure of 12,590 pounds to the square inch before cracking, and the other did not commence to crack until the pressure had reached 12,185 pounds to the square inch. The ultimate strength of the first block was 12,619 pounds, and of the second 12,874 pounds, per square inch.

Strength and Weight of Colorado Building Stones.

The following are the most reliable data obtainable of the strength and weight of the stones most extensively used for building in Colorado.

* *Red Granite from Platte Cañon.* Crushing strength per square inch, 14,600 pounds. Weight per cubic foot, 164 pounds.

Red Sandstone from Pike's Peak Quarry, Manitou. Crushing strength, 6.000 pounds per square inch.

** *Red Sandstone* from Greenlee & Son's quarries, Manitou (adjacent to the Pike's Peak quarries). Crushing weight, 11,000 pounds per square inch on bed. Weight, 140 pounds per cubic foot.

* *Gray Sandstone from Trinidad.* Crushing weight, 10.000 pounds per square inch. Weight, 145 pounds per cubic foot.

† *Lava Stone, Curry's Quarry, Douglas County.* Crushing strength, 10,675 pounds per square inch. Weight, 119 pounds per cubic foot. (Experience has shown that this stone is not suitable for piers, or where any great strength is required, as it cracks very easily.)

* *Fort Collins*, gray sandstone (laminated), much used for foundations.

Crushing strength, bed 11,700 pounds, edge 10,700 pounds per square inch Weight, 140 pounds per cubic foot. (One ton of this stone measures just a perch in the wall.)

* *St. Vrains*, light red sandstone (laminated), excellent stone for foundations. Very hard.

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* From tests made for the Board of Capitol Managers (of Colorado) by State Engineer E. S. Nettleton, in 1885, on two-inch cubes.

† From tests made by Denver Society of Civil Engineers, in 1884, also on two-inch cubes. ** Tested at U. S. Arsenal, Watertown, Mass.

Crushing strength, bed 11,505 pounds, edge 17,181 pounds per square inch. Weight, 150 pounds per cubic foot.

Effects of Freezing on Mortar.—Both cement and lime-mortar, mixed with salt, have been used in freezing weather without any bad effects. (See *American Architect*, vol. xxi., p. 283.)

Rule for the proportion of salt said to have been used in the works at Woolwich Arsenal: "Dissolve one pound of rock-salt in eighteen gallons of water when the temperature is at 32 degrees Fahr., and add three ounces of salt for every three degrees of lower temperature."

Durability of Hoop Iron Bond.—I believe that, embedded in lime-mortar at such depth as to protect it from the air, hoop iron bond is indestructible. In cement mortar containing salts of potash and soda, I doubt its lasting 1,500 years uncorroded. —M. C. Meigs, *May* 17, 1887.

Grouting.*

It is contended by persons having large experience in building that masonry carefully grouted, when the temperature is not lower than 40° Fahr., will give the most efficient result.

The following buildings in New York City have grouted walls:

Metropolitan Opera House.
Produce and Cotton Exchanges.
Mortimer and Mills Buildings.
Equitable and Mutual Life Insurance Buildings.
Standard Oil Building.
Astor Building.
The Eden Musée.
The Navarro Buildings.
Manhattan Bank Building.

The Presbyterian, German, St. Vincent, and Woman's Hospitals. etc; also, the Mersey Docks and Warehouses at Liverpool, England, one of the greatest pieces of masonry in the world, have been grouted throughout. It should be stated, however, that there are many engineers and others who do not believe in grouting, claiming that there is a tendency of the materials to separate and form layers.

* See American Architect, July 21, 1887, p. 11.

Architectural Terra-Cotta—Weight and Strength.

The lightness of terra-cotta, combined with its enormous resisting strength, and taken in connection also with its durability and absolute indestructibility by fire, water, frost, etc., renders it specially desirable for use in the construction of all large edifices.

Terra-cotta for building purposes, whether plain or ornamental, is generally made of hollow blocks formed with webs inside, so as to give extra strength and keep the work true while drying. This is necessitated because good, well-burned terra-cotta cannot safely be made of more than about 1½ inches in thickness, whereas, when required to bond with brick-work, it must be at least four inches thick. When extra strength is needed, these hollow spaces are filled with concrete or brick-work, which greatly increases the crushing strength of terra-cotta, although alone it is able to bear a very heavy weight. "A solid block of terra-cotta of one foot cube has borne a crushing strain of 500 tons and over."

Some exhaustive experiments, made by the Royal Institute of British Architects, give the following results as the crushing strength of terra-cotta blocks :

	Crushing wt. per cu. ft.
1. Solid block of terra-cotta	523 tons.
2. Hollow block of terra-cotta, unfilled	186 "
3. Hollow block of terra-cotta, slightly made and unfilled.	80 "

Tests of terra-cotta manufactured by the New York Company, which were made at the Stevens Institute of Technology in April, 1888, gave the following results :

	Crushing wt. per cu. in.	Crushing wt. per cu. ft.
Terra-cotta block, 2-inch square, red	6,840 lbs.	or 492 tons.
Terra-cotta block, 2-inch square, buff	6,236 "	" 449 "
Terra-cotta block, 2-inch square, gray	5,126 "	" 369 "

From these results, the writer would place the safe working strength of terra-cotta blocks in the wall at 5 tons per square foot when *unfilled*, and 10 tons per square foot when *filled solid* with brick-work or concrete.

The weight of terra-cotta in *solid blocks* is 122 pounds. When

made in hollow blocks 1½ inches thick, the weight varies from 6[illegible] to 85 pounds per cubic foot, the smaller pieces weighing the most. For pieces 12″ x 18″ or larger on the face, 70 pounds per cubic foot will probably be a fair average.

For the exterior facing of fire-proof buildings, terra-cotta is now considered as the most suitable material available.

CHAPTER VII.

STABILITY OF PIERS AND BUTTRESSES.

A PIER or buttress may be considered stable when the forces acting upon it do not cause it to rotate or "tip over," or any course of stones or brick to slide on its bed. When a pier has to sustain only a vertical load, it is evident that the pier must be stable, although it may not have sufficient strength.

It is only when the pier receives a thrust such as that from a rafter or an arch, that its stability must be considered.

In order to resist rotation, we must have the condition that the moment of the thrust of the pier about any point in the outside of the pier shall not exceed the moment of the weight of the pier about the same point.

To illustrate, let us take the pier shown in Fig. 1.

Let us suppose that this pier receives the foot of a rafter, which exerts a thrust T in the direction AB. The tendency of this thrust will be to cause the pier to rotate about the outer edge b_1; and the moment of the thrust about this point will be $T \times a_1b_1$, a_1b_1 being the arm. Now, that the pier shall be just in equilibrium, the moment of the weight of the pier about the same edge must just equal $T \times a_1b_1$. The weight of the pier will, of course, act through the centre of gravity of the pier (which in this case is at the centre), and in a vertical direction; and its arm will be b_1c, or one-half the thickness of the pier.

Hence, to have equilibrium, we must have the equation,

$$T \times a_1b_1 = W \times b_1c.$$

But under this condition the least additional thrust, or the crushing off of the outer edge, would cause the pier to rotate: hence, to have the pier in safe equilibrium, we must use some factor of safety.

This is generally done by making the moment of the weight equal to that of the thrust when referred to a point in the bottom of the pier, a certain distance in from the outer edge.

This distance for piers or buttresses should not be less than one-fourth of the thickness of the pier.

Representing this point in the figure by b, we have the necessary equation for the safe stability of the pier,

$$T \times ab = W \times \tfrac{1}{4}t,$$

t denoting the width of the pier.

We cannot from this equation determine the dimensions of a pier to resist a given thrust; because we have the distance ab, t, and W, all unknown quantities. Hence, we must first guess at the size of the pier, then find the length of the line ab, and see if the moment of the pier is equal to that of the thrust. If it is not, we must guess again.

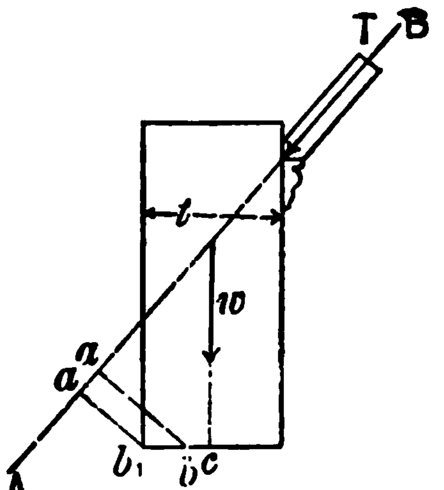

Fig. 1

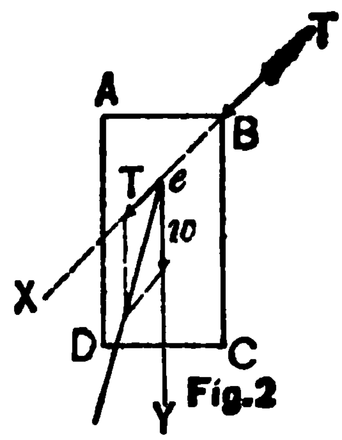

Fig. 2

Graphic Method of determining the Stability of a Pier or Buttress.—When it is desired to determine if a given pier or buttress is capable of resisting a given thrust, the problem can easily be solved graphically in the following manner.

Let $ABCD$ (Fig. 2) represent a pier which sustains a given thrust T at B.

To determine whether the pier will safely sustain this thrust, we proceed as follows.

Draw the indefinite line BX in the direction of the thrust. Through the centre of gravity of the pier (which in this case is at the centre of the pier) draw a vertical line until it intersects the line of the thrust at e. As a force may be considered to act anywhere in its line of direction, we may consider the thrust and the weight to act at the point e; and the resultant of these two forces can be obtained by laying off the thrust T from e on eX, and the weight of the pier W, from e on the line eY, both to the same scale (pounds to the inch), completing the parallelogram, and drawing the diagonal. If this diagonal prolonged cuts the base of the pier at less than one-fourth of the width of the base from the outer edge, the pier will be unstable, and its dimensions must be changed.

The stability of a pier may be increased by adding to its weight

(by placing some heavy material on top), or by increasing its width at the base, by means of "set-offs," as in Fig. 3.

Figs. 3 (A and B) show the method of determining the stability of a buttress with offsets.

The first step is to find the vertical line passing through the centre of gravity of the whole pier. This is best done by dividing the buttress up into quadrilaterals, as $ABCD$, $DEFG$, and $GHIK$ (Fig. 3A), finding the centre of gravity of each quadrilateral by the method of diagonals, and then measuring the perpendicular distances X_1, X_2, X_3, from the different centres of gravity to the line KI.

Multiply the area of each quadrilateral by the distance of its centre of gravity from the line KI, and add together the areas and the products. Divide the sum of the latter by the sum of the former, and the result will be the distance of the centre of gravity of the whole buttress from KI. This distance we denote by X_0.

Fig. 3A

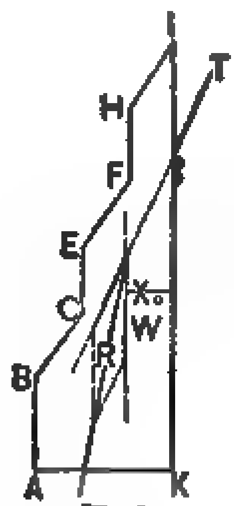

Fig. 3B

EXAMPLE I.—Let the buttress shown in Fig. 3A have the dimensions given between the cross-marks. Then the area of the quadrilaterals and the distances from their centres of gravity to KI would be as follows:

1st area = 35 sq. ft.	$X_1 = 0'.95$	1st area × X_1 = 33.25
2d area = 23 sq. ft.	$X_2 = 2'.95$	2d area × X_2 = 67.85
3d area = 11 sq. ft.	$X_3 = 4'.95$	3d area × X_3 = 54.45
Total area, 69 sq. ft.		Total moments, 155.55

The sum of the moments is 155.55; and, dividing this by the total area, we have 2.25 as the distance X_0. Measuring this to the scale of the drawing from KI, we have a point through which the vertical line passing through the centre of gravity must pass.

After this line is found, the method of determining the stability of the pier is the same as that given for the pier in Fig. 2. Fig. 3B also illustrates the method. If the buttress is more than one foot thick (at right angles to the plane of the paper), the cubic contents of the buttress must be obtained to find the weight. It is easier, however, to divide the real thrust by the thickness of the buttress, which gives the thrust per foot of buttress.

Line of Resistance.—*Definition.* The line of resistance or of pressures, of a pier or buttress, is a line drawn through the centre of pressure of each joint.

The *centre of pressure* of any joint is the point where the resultant of the forces acting on that portion of the pier above the joint cuts it.

The line of pressures, or of resistance, when drawn in a pier, shows how near the greatest stress on any joint comes to the edges of that joint.

It can be drawn by the following method.

Let $ABCD$ (Fig. 4) be a pier whose line of resistance we wish to draw. First divide the pier in height, into portions two or three feet high, by drawing horizontal lines. It is more convenient to make the portions all of the same size.

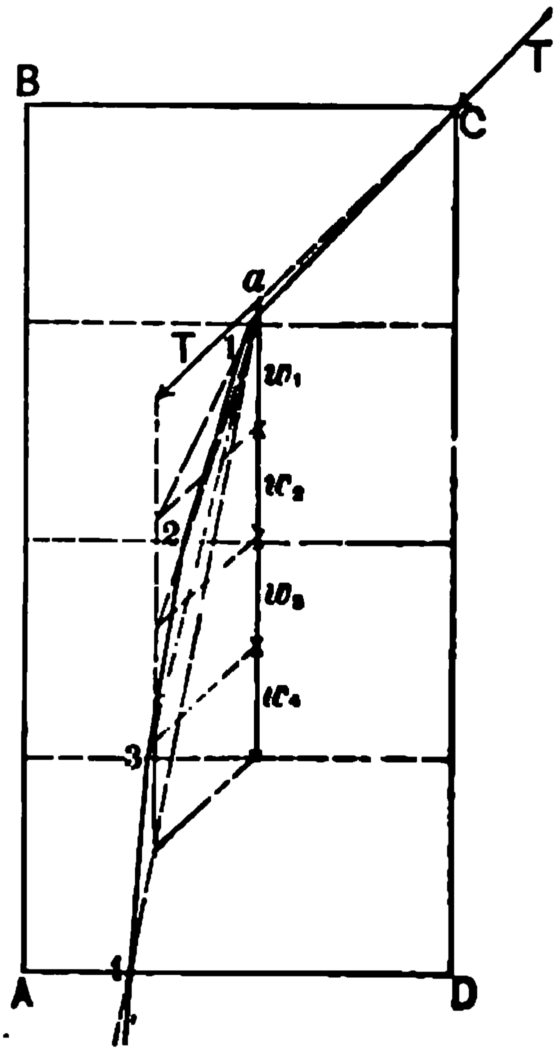

Fig. 4.

Prolong the line of the thrust, and draw a vertical line through the centre of gravity of the pier, intersecting the line of thrust at the point a. From a lay off to a scale the thrust T and the weights of the different portions of the pier, commencing with the weight of the upper portion. Thus, w_1 represents the weight of the portion above the first joint; w_2 represents the weight of the second portion; and so on. The sum of the w's will equal the whole weight of the pier.

Having proceeded thus far, complete a parallelogram, with T and w_1 for its two sides. Draw the diagonal, and prolong it. Where it cuts the first joint will be a point in the line of resistance. Draw another parallelogram, with T and $w_1 + w_2$ for its two sides. Draw the diagonal intersecting the second joint at 2. Proceed in

[illegible]

the center of gravity [illegible] distance [illegible] from the line [illegible] Fig. [illegible]. Then, on Fig. [illegible] measure the distance [illegible] and through the point [illegible] draw a vertical line intersecting the line [illegible]

[illegible] Now if the thrust is [illegible] pounds [illegible] it would be half that, or [illegible] pounds.

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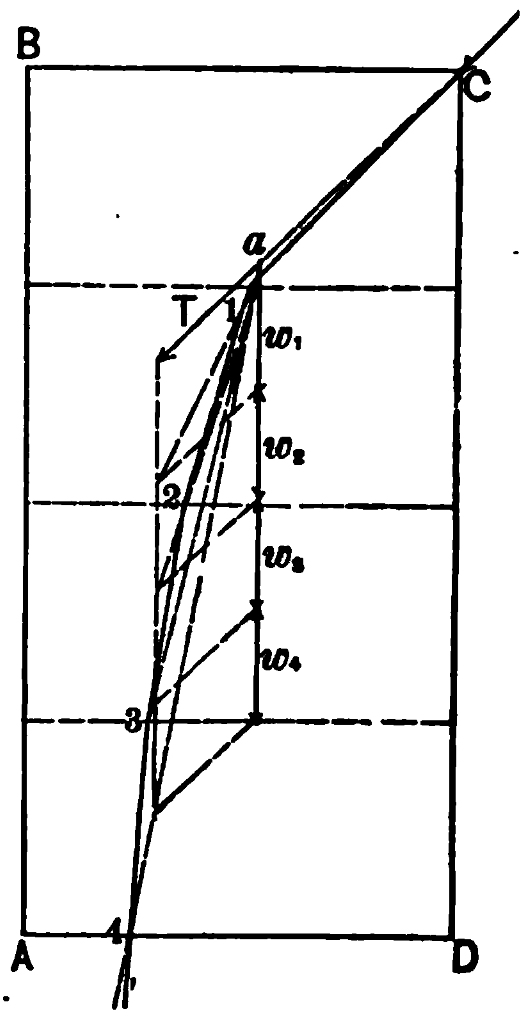

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this way, when the last diagonal will intersect the base in 4. Join the points 1, 2, 3, and 4, and the resulting line will be the line of resistance.

We have taken the simplest case as an example; but the same principle is true for any case.

Should the line of resistance of a pier at any point approach the outside edge of the joint nearer than one-quarter the width of the joint, the pier should be considered unsafe.

As an example embracing all the principles given above, we will take the following case.

EXAMPLE II. — Let Fig. 5 represent the section of a side wall of a church, with a buttress against it. Opposite the buttress, on the inside of the wall, is a hammer-beam truss, which we will suppose exerts an outward thrust on the walls of the church amounting to about 9600 pounds. We will further consider that the resultant of the thrust acts at P, and at an angle of 60° with a horizontal. The dimensions of the wall and buttress are given in Fig. 5A, and the buttress is two feet thick.

QUESTION. — Is the buttress sufficient to enable the wall to withstand the thrust of the truss?

The first point to decide is if the line of resistance cuts the joint CD at a safe distance in from C. To ascertain this, we must find the centre of gravity of the wall and buttress above the joint CD. We can find this easiest by the method of moments around KM (Fig. 5A), as already explained.

The distance X_1 is, of course, half the thickness of the wall, or one foot. We next find the centre of gravity of the portion $CEFG$ (Fig. 5A), by the method of diagonals, and, scaling the distance X_2, we find it to be 2.95 feet.

The area of $CEFG = A_2 = 10$ square feet; and of $GIKL = A_1 = 26$ square feet.

Then we have,

$$\begin{array}{lll} X_1 = 1 & A_1 = 26 & A_1 \times X_1 = 26 \\ X_2 = 2.95 & A_2 = \underline{10} & A_2 \times X_2 = \underline{29.5} \\ & 36 & 36\,)\,55.5 \\ & & X_0 = \overline{1.5} \end{array}$$

Or the centre of gravity is at a distance 1.5 foot from the line ED (Fig. 5). Then on Fig. 5 measure the distance $X_0 = 1.5$ foot, and through the point a draw a vertical line intersecting the line of the thrust prolonged at O. Now, if the thrust is 9600 pounds for a buttress two feet thick, it would be half that, or 4800 pounds, for a buttress one foot thick. We will call the weight of the

masonry of which the buttress and wall is built 150 pounds per cubic foot. Then the thrust is equivalent to 4800 ÷ 150, or 32 cubic feet of masonry. Laying this off to a scale from O, in the direction of the thrust and the area of the masonry, 36 square feet from O on the vertical line, completing the rectangle, and drawing the diagonal, we find it cuts the joint CD at b, within the limits of safety.

We must next find where the line of resistance cuts the base AB.

Fig. 5A

First find the centre of gravity of the whole figure, which is found by ascertaining the distances X_2', X_3', in Fig. 5A, and making the following computation:

$$
\begin{array}{lll}
X_1' = 1' & A_1' = 40 & A_1' \times X_1' = 40 \\
X_2' = 2'.98 & A_2' = 24 & A_2' \times X_2' = 71.52 \\
X_3' = 4'.95 & A_3' = \underline{12} & A_3' \times X_3' = \underline{59.40} \\
 & \phantom{A_3' = {}}76 & \phantom{A_3' \times X_3' = {}}76\,)\,170.92 \\
 & & \phantom{A_3' \times {}}X_0' = 2.25
\end{array}
$$

Then from the line EB (Fig. 5) lay off the distance $X_0' = 2'.25$, and draw through d a vertical line intersecting the line of the thrust at O'. On this vertical from O' measure down the whole area 76, and from its extremity lay off the thrust $T = 32$ at the

proper angle. Draw the line $O'c$ intersecting the base at c. This is the point where the line of resistance cuts the base; and, as it is at a safe distance in from A, the buttress has sufficient stability.

If there were more offsets, we should proceed in the same way, finding where the line of resistance cuts the joint at the top of each offset. The reason for doing this is because the line of resistance might cut the base at a safe distance from the outer edge, while higher up it might come outside of the buttress, so that the buttress would be unstable.

The method given in these examples is applicable to piers of any shape or material.

Should the line of resistance make an angle less than 30° with any joint, it might cause the stones above the joint to slide on their bed. This can be prevented either by dowelling, or by inclining the joint.

It is very seldom in architectural construction that such a case would occur, however.

CHAPTER VIII.

THE STABILITY OF ARCHES.

The arch is an arrangement for spanning large openings by means of small blocks of stone, or other material, arranged in a particular way. As a rule, the arch answers the same purpose as the beam, but it is widely different in its action and in the effect that it has upon the appearance of an edifice. A beam exerts merely a vertical force upon its supports, but the arch exerts both a vertical load and an outward thrust. It is this thrust which requires that the arch should be used with caution where the abutments are not abundantly large.

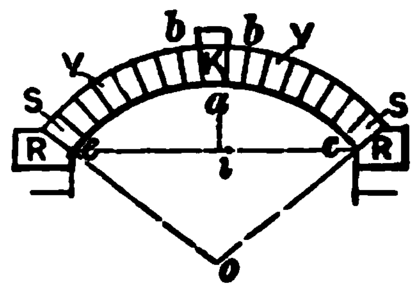

Fig. 1

Before taking up the principles of the arch, we will define the many terms relating to it. The distance *ec* (Fig. 1) is called the *span* of the arch; *ai*, its rise; *b*, its *crown*; its lower boundary line, *eac*, its *soffit* or *intrados*; the outer boundary line, its *back* or *extrados*. The terms "soffit" and "back" are also applied to the entire lower and upper curved *surfaces* of the whole arch. The ends of the arch, or the sides which are seen, are called its *faces*. The blocks of which the arch itself is composed are called *voussoirs*: the centre one, *K*, is called the *keystone*; and the lowest ones, *SS*, the *springers*. In *segmental* arches, or those whose intrados is not a complete semicircle, the springers generally rest upon two stones, as RR, which have their upper surface cut to receive them: these stones are called *skewbacks*. The line connecting the lower edges of the springers is called the *springing-line*; the sides of the arch are called the *haunches*; and the load in the triangular space, between the haunches and a horizontal line drawn from the crown, is called the *spandrel*.

The blocks of masonry, or other material, which support two successive arches, are called *piers*: the extreme blocks, which, in the case of stone bridges, generally support on one side embankments of earth, are called *abutments*.

A pier strong enough to withstand the thrust of either arch, should the other fall down, is sometimes called an *abutment pier*. Besides their own weight, arches usually support a permanent load or surcharge of masonry or of earth.

In using arches in architectural constructions, the form of the

arch is generally governed by the style of the edifice, or by a limited amount of space. The semicircular and segmental forms of arches are the best as regards stability, and are the simplest to construct. Elliptical and three-centred arches are not as strong as circular arches, and should only be used where they can be given all the strength desirable.

The strength of an arch depends very much upon the care with which it is built and the quality of the work.

In stone arches, special care should be taken to cut and lay the beds of the stones accurately, and to make the bed-joints thin and close, in order that the arch may be strained as little as possible in settling.

To insure this, arches are sometimes built dry, *grout* or liquid mortar being afterwards run into the joints; but the advantage of this method is doubtful.

Brick Arches may be built either of wedge-shaped bricks, moulded or rubbed so as to fit to the radius of the soffit, or of bricks of common shape. The former method is undoubtedly the best, as it enables the bricks to be thoroughly bonded, as in a wall; but, as it involves considerable expense to make the bricks of the proper shape, this method is very seldom employed. Where bricks of the ordinary shape are used, they are accommodated to the curved figure of the arch by making the bed-joints thinner towards the intrados than towards the extrados; or, if the curvature is sharp, by driving thin pieces of slate into the outer edges of those joints; and different methods are followed for bonding them. The most common way is to build the arch in concentric rings, each half a brick thick; that is, to lay the bricks all stretchers, and to depend upon the tenacity of the mortar or cement for the connection of the several rings. This method is deficient in strength, unless the bricks are laid in cement at least as tenacious as themselves. Another way is to introduce courses of headers at intervals, so as to connect pairs of half-brick rings together.

This may be done either by thickening the joints of the outer of a pair of half-brick rings with pieces of slate, so that there shall be the same number of courses of stretchers in each ring between two courses of headers, or by placing the courses of headers at such distances apart, that between each pair of them there shall be one course of stretchers more in the outer than in the inner ring.

The former method is best suited to arches of long radius; the latter, to those of short radius. *Hoop iron* laid round the arch, between half-brick rings, as well as longitudinally and radially, is very useful for strengthening brick arches. The bands of hoop iron which traverse the arch radially may also be bent, and prolonged in the bed-joints of the backing and spandrels.

By the aid of hoop-iron bond, Sir Marc-Isambard Brunel half-arch of bricks laid in strong cement, which stood, pr from its abutment like a bracket, to the distance of sixty fe it was destroyed by its foundation being undermined.

The New-York City Building Laws make the following ments regarding brick arches: —

"All arches shall be at least four inches thick. Arches o foot span shall be increased in thickness toward the haun additions of four inches in thickness of brick. The first ad thickness shall commence at two and a half feet from the c the span; the second addition, at six and one-half feet from t tre of the span; and the thickness shall be increased then inches for every additional four feet of span towards the ha

"The said brick arches shall be laid to a line on the centr a close joint, and the bricks shall be well wet, and the join with cement mortar in proportions of not more than two to one of cement by measure. The arches shall be well and pinned, or chinked with slate, and keyed."

Rule for Radius of Brick Arches. — A good rule for the of segmental brick arches over windows, doors, and othe openings, is to *make the radius equal to the width of the o* This gives a good rise to the arch, and makes a pleasing proportion to the eye.

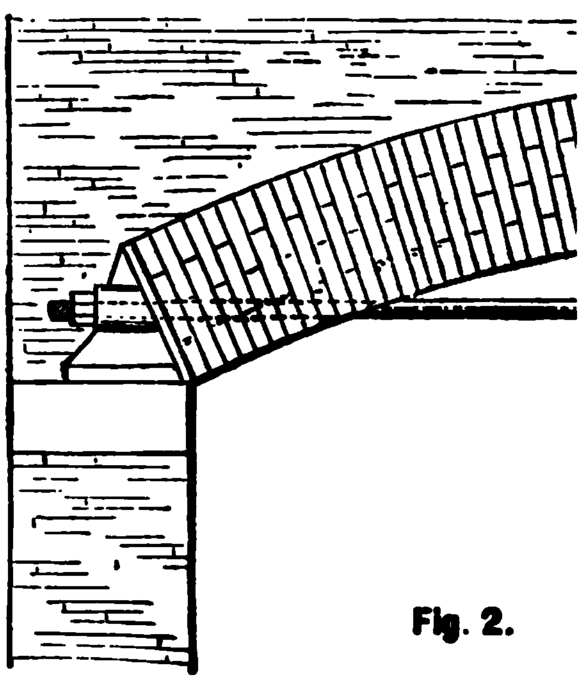

Fig. 2.

It is often desirable to span openings in a wall by means of an arch, when there is not sufficient abutments to withstand the thrust or kick of the arch. In such a case, the arch can be formed on two cast-iron skewbacks, which are held in place by iron rods, as is shown in Fig. 2.

When this is done, it is necessary to proportion the size rods to the thrust of the arch. The horizontal thrust of the very nearly represented by the following formula: —

$$\text{Horizontal thrust} = \frac{\text{load on arch} \times \text{span}}{8 \times \text{rise of arch in feet}}.$$

If two tension rods are used, as is generally the case, the ter of each rod can be determined by the following rule: —

$$\text{Diameter in inches} = \sqrt{\frac{\text{total load on arch} \times \text{span}}{16 \times \text{rise of arch in feet} \times 78}}$$

If only one rod is used, 8 should be substituted in the place of 16, in the denominator of the above rule; and, if three rods are used, 24 should be used instead of 16.

Centres for Arches. — *A centre* is a temporary structure, generally of timber, by which the voussoirs of an arch are supported while the arch is being built. It consists of parallel frames or ribs, placed at convenient distances apart, curved on the outside to a line parallel to that of the soffit of the arch, and supporting a series of transverse planks, upon which the arch stones rest.

The most common kind of centre is one which can be lowered, or struck all in one piece, by driving out wedges from below it, so as to remove the support from every point of the arch at once.

The centre of an arch should not be struck until the solid part of the backing has been built, and the mortar has had time to set and harden; and, when an arch forms one of a series of arches with piers between them, no centre should be struck so as to leave a pier with an arch abutting against one side of it only, unless the pier has sufficient stability to act as an abutment.

When possible, the centre of a large brick arch should not be struck for two or three months after the arch is built.

Mechanical Principles of the Arch. — In designing an arch, the first question to be settled is the form of the arch; and in regard to this there is generally but little choice. Where the abutments are abundantly large, the segmental arch is the strongest form; but, where it is desired to make the abutments of the arch as light as possible, a pointed or semicircular arch should be used.

Depth of Keystone. — Having decided upon the form of the arch, the depth of the arch-ring must next be decided. This is generally determined by computing the required depth of keystone, and making the whole ring of the same or a little larger depth.

In considering the strength of an arch, the depth of the keystone is considered to be only the distance from the extrados to the intrados of the arch; and if the keystone projects above the arch-ring, as in Fig. 1, the projection is considered as a part of the load on the arch.

There are several rules for determining the depth of the keystone, but all are empirical; and they differ so greatly that it is difficult to recommend any particular one. Professor Rankine's Rule is often quoted, and is probably true enough for most arches. It applies to both circular and elliptical arches, and is as follows: —

Rankine's Rule. — *For the depth of the keystone*, take a mean proportional between the inside radius at the crown, and 0.12 of a foot for a single arch, and 0.17 of a foot for an arch forming one of a series. Or, if represented by a formula,

But, if we should compute the stability of a semicircular arch of 20 foot span, and 1.3 foot depth of keystone, we should find that the arch was very unstable: hence, in this case, we must throw the rule aside, and go by our own judgment. In the opinion of the author, such an arch should have at least 2½ feet depth of arch-ring, and we will try the stability of the arch with that thickness.

In all calculations on the arch, it is customary to consider the arch to be one foot thick at right angles to its face; for it is evident, that, if an arch one foot thick is stable, any number of arches of the same dimensions built alongside of it would be stable.

Graphic Solution of the Stability of the Arch.— The most convenient method of determining the stability of the arch is by the graphic method, as it is called.

1st Step. — Draw one-half the arch to as large a scale as convenient, and divide it up into voussoirs of equal size. In this example, shown in Fig. 3, we have divided the arch-ring into ten equal voussoirs. (It is not necessary that these should be the actual voussoirs of which the arch is built.) The next step is to find the area of each voussoir. Where the arch-ring is divided into voussoirs of equal size, this is easiest done by computing the area of the arch-ring, and dividing by the number of voussoirs.

Fig.3

Rule for area of one-half of arch-ring is as follows:—

Area in square feet = 0.7854 × (outside radius squared — inside radius squared).

In this example the whole area equals $0.7854 \times (12.5^2 - 10^2) =$ 44.2 square feet. As there are ten equal voussoirs, the area of each voussoir is 4.4 square feet.

Having drawn out one-half of the arch-ring, we divide each joint into three equal parts; and from the point A (Fig. 3) we lay off to a scale the area of each voussoir, one below the other, commencing

with the top voussoir. The whole length of the line AE will equal the whole area drawn to same scale.

The next step is to find the vertical line passing through the centre of gravity of the whole arch-ring. To do this, it is first necessary to draw vertical lines through the centre of gravity of each voussoir. The centre of gravity of one voussoir may be found by the method of diagonals, as in the second voussoir from the top (Fig. 3). Having the centre of gravity of one voussoir, the centres of gravity of the others can easily be obtained from it.

Next, from A and E (Fig. 3) draw lines at 45° with AE, intersecting at O. Draw $O1$, $O2$, $O3$, etc. Then, where AO intersects the first vertical line at a, draw a line parallel to $O1$, intersecting the second vertical at b. Draw bc parallel to $O2$, cd parallel to $O3$, and so on to kn parallel to $O10$: prolong this line downward until it intersects AO, prolonged at D. Then a vertical line drawn through D will pass through the centre of gravity of the arch-ring.

2D STEP. — Draw a horizontal line through A (the upper part of the middle third), and a vertical line through D; the two lines intersecting at C (Fig. 3).

Now, that the arch shall be stable, it is considered necessary that it shall be possible to draw a line of resistance of the arch within the middle third. We will, then, first assume that the line of resistance shall act at A, and come out at B.

Then draw the line CB, and a horizontal line opposite the point 10, between Q and P. This horizontal line represents the horizontal thrust at the crown.

Draw AP equal to QP, and the lines $P1$, $P2$, $P3$, etc.

Then, from the point where AC prolonged intersects the first vertical, draw a line to the second vertical, parallel to $P1$; from this point a line to the third vertical, parallel to $P2$; and so on. The last line should pass through B. If these lines, which we will call the line of resistance, all lie within the middle third, the arch may be considered to be stable. Should the line of resistance pass outside of the arch-ring, the arch should be considered unstable. In Fig. 3 this line does not all lie in the middle third, and we must see if a line of resistance can yet be drawn within that limit.

2D TRIAL. — The line of resistance in Fig. 3 passes farthest from the middle third at the seventh joint from the top; and we will next pass a line of resistance through A and where the lower line of the middle third cuts the seventh joint, or at D (Fig. 4).

To do this, we must prolong the line gh, parallel to $O7$ (Fig. 4), until it intersects AO. In this case it intersects it at O: but this is merely a coincidence; it would not always do so. Through O draw a vertical intersecting PA prolonged at C. Draw a line

through C and D, and the horizontal line pQ, opposite the point 7: this line represents the new horizontal thrust H_1. Draw $AP = pQ$, and the lines $P1$, $P2$, etc.; then draw the line of resistance as before. It should pass through D if drawn correctly. This time we see that the line of resistance lies within the middle third, except just a short distance at the springing; and hence we may consider the arch stable. If it had gone outside the middle third this time, to any great extent, we should have considered the arch unstable.

The above is the method of determining the stability of an unloaded semicircular arch. Such a case very seldom occurs in practice; but it is a good example to illustrate the method, which applies to all other cases, with a little difference in the method of determining the centre of gravity of loaded arches.

Fig.4

EXAMPLE II. — *Loaded or surcharged semicircular arch.*

We will take the same arch as in Example I., and suppose it to be loaded with a wall of masonry of the same thickness and weight per square foot as that of the arch-ring; the horizontal surface of the wall being 3 feet 6 inches above the arch-ring at the crown.

1ST STEP. — *Find centre of gravity.*

Commencing at the crown, divide the load and arch-ring into strips two feet wide, making the last strip the width of the arch-ring at the springing. Then draw the joints as shown in Fig. 5. Measure with the scale the length of each vertical line, Aa, Bb, etc.; then the area of $AaBb$ is equal to the length of $Aa + Bb$, as the distance between them is just two feet. The area of $FfKk$ is, of course, $Ff \times$ width of arch-ring.

In this case, the areas of the slices are as shown by the figures on their faces (Fig. 5).

Now divide the arch-ring into thirds, and from the top of the middle third, at R, lay off in succession, to a scale, the areas of

slices, commencing with the first slice from the crown, $AaBb$. se areas, when measured off, will be represented by the line 2, 3 . . . 6 (Fig. 5). From the extremities of this line, R and 6, v lines at 45° with a vertical, intersecting at O. From O draw s to 1, 2, 3, 4, 5, and 6. Next, draw a vertical line through the re of each slice (these lines, in Fig. 5, are numbered 1, 2, 3,). From the point in which the line RO intersects vertical 1, v a line parallel to $O1$, to the line 2. From this point draw a to vertical 3, parallel to $O2$, and so on. The line parallel to will intersect vertical 6 at Y. Then through Y draw a line nwards at 45°, intersecting OR at X. A vertical line drawn ugh X will pass through the centre of gravity of the arch-ring its load.

STEP. — *To find the thrust at the crown and at the springing.* o find the thrust at the crown, draw a vertical line through X, a horizontal line through R, intersecting at V. Now, the weight ch and load, and the resultant thrust of arch, must act through point. We will also make the condition that the thrust shall through Q, the outer edge of the middle third. Then the st of the arch must act in the line VQ. Opposite 6, on the ical line through R, draw a horizontal line H, between VX VQ. This horizontal line represents a horizontal thrust at R, ch would cause the resultant thrust of the arch to pass through Now draw the horizontal line RP, equal in length to H, and n P draw lines 1, 2, 3 . . . 6. The line $P6$ represents the thrust he arch at the springing. Its amount in cubic feet of masonry be determined by measuring its length to the proper scale.

3D STEP. — *To draw the line of resistance.*

The lines $P1$, $P2$, $P3$, etc., represent the magnitude and direction of the thrust at each joint of the arch. Thus $P1$ represents the thrust of the first voussoir and its load; $P2$, that of the first two voussoirs and their loads; and so on. Then from the point a', where the line RP, prolonged, intersects the vertical line 1, draw a line $a'b'$ parallel to $P1$; from b', on 2, draw a line $b'c'$ parallel to $P2$, and so on. The last line should pass through Q, and be parallel to $P6$.

Now, if we connect the points where the lines $a'b'$, $b'c'$, etc., cut the joints of the arch, we shall have a broken line, which is known as the line of resistance of the arch. If this line lies within the middle third of the arch, then we conclude that the arch is stable. If the line of resistance goes far outside of the middle, we must see if it be possible to draw another line of resistance within the middle third; and if, after a trial, we find that it is not possible, we must conclude that the arch is not safe, or unstable.

In the example which we have just been discussing, the line of resistance goes a little outside of the middle third; but it is very probable that on a second trial we should find that a line of resistance passed through R and Q' would lie almost entirely within the middle third.

The method of drawing the second line of resistance was explained under Example I.; and, as the same method applies to all cases, we will not repeat it.

The method given for Example II. would apply equally well for a semi-elliptical arch.

EXAMPLE III. — *Segmental arch, with load* (Fig. 6).

1ST STEP. — *To determine the centre of gravity.*

In this case we proceed, the same as in the latter, to divide the arch-ring and its load into vertical slices two feet wide, and compute the area of the slices by measuring the length of the vertical lines Aa, Bb, etc. Having computed the areas of the slices, we lay them off in order from R, to a convenient scale, and then proceed exactly as in Example II., the remaining steps determining the thrust; and the lines of resistance are also the same as given under Example II.

In a flat segmental arch, there is practically no need of dividing the arch-ring into voussoirs by joints radiating from a centre, but to consider the joints to be vertical. Of course, when built, they must be made to radiate.

Fig. 6 shows the computation for an arch of 40-foot span, and with a load $13\frac{1}{2}$ feet high at the centre. The depth of the arch-ring is 2 feet 6 inches.

It will be seen, that the curve of pres[sure] lies e[nt]irely within

iddle third; and hence the arch is abundantly safe, or stable. uld be remarked, that the line of resistance in a segmental should be drawn through the *lower* edge of the middle third springing.

D.

ill be noticed that the horizontal thrust, and the thrust T, springing, are very great as compared with those in a semi- ar arch; and hence, although the segmental arch is the er of the two, it requires much heavier abutments.

se three examples serve to show the method of determining bility and thrust of any arch such as is used in building.

CHAPTER IX.

RESISTANCE TO TENSION,

OR THE STRENGTH OF TIE-RODS, BARS, ROPES, AND CHAINS.

THE resistance which any material offers to being pulled apart is due to the tenacity of its fibres, or the cohesion of the particles of which it is composed.

It is evident that the amount of resistance to tension which any cross-section of a body will exert depends only upon the tenacity of its fibres, or the cohesion of its particles, and upon the number of fibres, or particles, in the cross-section.

As the number of the fibres, or particles, in the section, is proportional to the area, the strength of any piece of material must be as the area of its cross-section; and hence, if we know the tenacity of the material per square inch of cross-section, we can obtain the total strength by multiplying it by the area of the section in inches.

The tenacity of different building-materials per square inch has been found by pulling apart a bar of the material of known dimensions, and dividing the breaking-force by the area of the cross-section of the bar.

TABLE I. gives the average values for the tenacity of building-materials, as determined by the most reliable experiments.

Knowing the tenacity of one square inch of the material, all that is necessary to determine the tenacity of a piece of any uniform size is to multiply the area of its cross-section, in square inches, by the number in the table opposite the name of the material. This would give the weight that would just break the piece; but, as what we wish is the safe load, we must divide the result by a factor of safety. Most engineers advise using a factor of safety of five for a dead load, although the New-York City and also the Boston Building Laws require a factor of six.

Denoting the factor of safety by S, and the tenacity by T, we have as a rule,

For a rectangular bar,

$$\text{Safe load} = \frac{\text{breadth} \times \text{depth} \times T}{S} \qquad (1)$$

TABLE I.

Average Ultimate Tensile or Cohesive Strength of Materials.

For a round bar,

$$\text{Safe load} = \frac{0.7854 \times \text{diameter squared} \times T}{S} \qquad (2)$$

EXAMPLE I. — What is the safe load for a tie-bar of white pine 6 by 6 inches?

Ans. Here the breadth and depth both equal 6 inches, $T = 7000$, and we will let $S = 5$; then,

$$\text{Safe load} = \frac{6 \times 6 \times 7000}{5} = 50400 \text{ lbs.}$$

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[a] Trautwine. [b] Rankine. [c] Hodgkinson. [d] Kirkaldy. [e] Fairbairn. [f] United States Government, Watertown Arsenal. [g] Barlow. [h] Bevan. [i] Hatfield. [j] Rondelet.

the size of the bar is desired, we have,

$$\text{The breadth} = \frac{S \times \text{load}}{\text{depth} \times T} \quad (3)$$

For a round bar,

$$\text{Diameter squared} = \frac{S \times \text{load}}{0.7854 \times T} \quad (4)$$

EXAMPLE II. — It is desired to suspend 20,000 pounds from a round rod of wrought-iron: what shall be the diameter of the rod to carry the weight in safety?

Ans. In this case $T = 50,000$; and taking S at 5, we have

$$\text{Diameter squared} = \frac{5 \times 20000}{0.7854 \times 50000} = 2.54.$$

The square root of this is 1.6 or $1\frac{5}{8}$ inches nearly: therefore the diameter of the rod should be $1\frac{5}{8}$ inches.

Tensile Strength and Qualities of Steel.

The elastic limit of steel should not be less than 40,000 pounds per square inch for high grade steel, 36,000 pounds for medium steel, and 30,000 pounds for soft steel.

The ultimate tensile strength of high grade steel should range between 70,000 and 80,000 pounds per square inch; of medium, between 60,000 and 70,000; and of soft steel, between 52,000 and 60,000 pounds per square inch.

The elongation in a length of 8 inches should be not less than 18 per cent. for high grade steel, 22 per cent. for medium, and 25 per cent. for soft steel.

The reduction of area at point of fracture should be not less than 35 per cent. of the original area.

High grade steel (35 per cent. carbon) should be used for compression, bolsters, bearing-plates, pins, and rollers.

Medium steel (15 per cent. carbon) should be used for tension members, floor system, laterals, bracing, and, unless high grade steel is specified, should be used for all steel members except rivets.

Soft steel (11 or 12 per cent. carbon) should be used in rivets only, and should be tested by actually making up into rivets, riveting two plates together, and upon being nicked and cut out should show a good, tough, silky structure, with no crystalline appearance. Rivet steel should not have over 0.15 per cent. carbon.

Steel made by the Bessemer process should not [illegible] over 0.08 per cent. of phosphorus, and open hearth steel [illegible] over $\frac{1}{10}$ of 1

per cent. The amount of phosphorus allowable should always be stated in the specifications, as this determines the price of the pig iron required to make the steel. About 0.04 per cent. of sulphur is allowable, and sometimes more.*

The Working Strength of steel in bridges is generally taken at 12,000 pounds per square inch, and in roof trusses, and structures sustaining a steady load, at 15,000 pounds per square inch; or, in a general way, the strength of steel is generally taken at 20 per cent. over that allowable for wrought iron under the same conditions.

STANDARD SPECIFICATION, ADOPTED BY BRIDGE-BUILDERS, FOR MATERIAL AND WORKMANSHIP OF IRON AND STEEL STRUCTURES.

QUALITY OF MATERIALS.

Wrought Iron.

Character and Finish.—1. All wrought iron must be tough, ductile, fibrous, and of uniform quality for each class, straight, smooth, free from cinder pockets or injurious flaws, buckles, blisters, or cracks. As the thickness of bars approaches the maximum that the rolls will produce, the same perfection of finish will not be required as in thinner ones.

2. No specific process or provision of manufacture will be demanded, provided the material fulfils the requirements of this specification.

Standard Test Piece.—3. The tensile strength, limit of elasticity and ductility, shall be determined from a standard test piece, not less than one quarter inch in thickness, cut from the full-size bar, and planed or turned parallel; if the cross-section is reduced, the tangent between shoulders shall be at least twelve times its shortest dimension, and the area of minimum cross-section in either case shall be not less than one-quarter of a square inch and not more than one square inch. Whenever practicable, two opposite sides of the piece are to be left as they come from the rolls, but the finish of opposite sides must be the same in this respect. A full-size bar, when not exceeding the above limitations, may be used as its own test piece. In determining the ductility the elongation shall be measured, after breaking, on an original length the nearest multiple of a quarter inch to ten times the shortest dimension of the test piece, in which length must occur the

* James Ritche, before the Civil Engineers' Club of Cleveland.

curve of reduction from stretch on both sides of the point of fracture, but in no case on a shorter length than five inches.

Tension Iron for Open Trusses.—4. All iron to be used in the tensile members of open trusses, laterals, pins and bolts, except plate iron over eight inches wide and shaped iron, must show by the standard test piece a tensile strength in pounds per square inch of :

$$52{,}000 - \frac{7{,}000 \times \text{area of original bar}}{\text{circumference of original bar}} \quad \text{(all in inches)},$$

with an elastic limit not less than one-half the strength given by this formula, and an elongation of twenty per cent.

Plate Iron.—5. Plate iron 24 inches wide and under, and more than 8 inches wide, must show by the standard test pieces a tensile strength of 48,000 pounds per square inch, with an elastic limit not less than 26,000 pounds per square inch, and an elongation of not less than 12 per cent. All plates over 24 inches in width must have a tensile strength not less than 46,000 pounds per square inch with an elastic limit not less than 26,000 pounds per square inch. Plates from 24 inches to 36 inches in width must have an elongation of not less than 10 per cent.; those from 36 inches to 48 inches in width, 8 per cent.; over 48 inches in width, 5 per cent.

Shaped Iron.—6. All shaped iron and other iron not hereinbefore specified must show by the standard test pieces a tensile strength in pounds per square inch of :

$$50{,}000 - \frac{7{,}000 \times \text{area of original bar}}{\text{circumference of original bar}},$$

with an elastic limit of not less than one-half the strength given by this formula, and an elongation of 15 per cent. for bars five-eighths of an inch and less in thickness, and of 12 per cent. for bars of greater thickness.

Hot Bending.—7. All plates, angles, etc., which are to be bent hot, in the manufacture must, in addition to the above requirements, be capable of bending sharply to a right angle at a working heat without sign of fracture.

Rivet Iron.—8. All rivet iron must be tough and soft, and pieces of the full diameter of the rivet must be capable of bending cold until the sides are in close contact without sign of fracture on the convex side of the curve.

Bending Tests.—9. All iron specified in clause 4 must bend cold, 180 degrees, without sign of fracture, to a curve the inner radius of which equals the thickness of the piece tested.

10. Specimens of full thickness cut from plate iron, or from the flanges or webs of shaped iron, must stand bending cold, through 90 degrees, to a curve the inner radius of which is one and a half times its thickness, without sign of fracture.

Number of Test Pieces.—11. For each contract four standard test pieces and one additional for each 50,000 pounds of wrought iron will, if required, be furnished and tested by the contractor without charge, and if any additional tests are required by the purchaser, they will be made for him at the rate of $5.00 each; or, if the contractor desires additional tests, they shall be made at his own expense, under the supervision of the purchaser, the quality of the material to be determined by the result of all the tests in the manner set forth in the following clause.

12. The respective requirements stated are for an average of the tests for each, and the lot of bars or plates from which samples were selected shall be accepted if the tests give such average results; but, if any test piece gives results more than 4 per cent. below said requirements, the particular bar from which it was taken may be rejected, but such tests shall be included in making the average. If any test piece has a manifest flaw, its test shall not be considered. For each bar thus giving results more than 4 per cent. below the requirements, tests from two additional bars shall be furnished by the contractor without charge, and if in a total of not more than ten tests, two bars (or, for a larger number of tests, a proportionately greater number of bars) show results more than 4 per cent. below the requirements, it shall be cause for rejecting the lot from which the sample bars were taken. Such lots shall not exceed 20 tons in weight, and bars of a single pattern, plates rolled in universal mill or in grooves, and sheared plates shall each constitute a separate lot.

Time of Inspection.—13. The inspection and tests of the material will be made promptly on its being rolled, and the quality determined before it leaves the rolling-mill. All necessary facilities for this purpose shall be afforded by the manufacturer; but, if the inspector is not present to make the necessary tests, after due notice given him, then the contractor shall proceed to make such number of tests on the iron then being rolled as may have been agreed upon; or, in the absence of any special agreement, the number provided for in clause 11, and the quality of such material shall be determined thereby.

Variation of Weight.—14. A variation in cross-section or weight of rolled material of more than $2\frac{1}{2}$ per cent. from that specified may be cause for rejection.

STEEL.

15. No specific process or provision of manufacture will be demanded, provided the material fulfils the requirements of this specification.

Test Bars.—16. From three separate ingots of each cast a round sample bar, not less than three-quarters of an inch in diameter, and having a length not less than twelve diameters between jaws of testing machine, shall be furnished and tested by the manufacturer without charge. These bars are to be truly round, and shall be finished at a uniform heat, and arranged to cool uniformly, and from these test pieces alone, the quality of the material shall be determined as follows :

Tensile Tests.—17. All the above described test bars must have a tensile strength within 4,000 pounds per square inch of that specified, an elastic limit not less than one-half of the tensile strength of the test bar, a percentage of elongation not less than 1,200,000 ÷ the tensile strength in pounds per square inch, and a percentage of reduction of area not less than 2,400,000 ÷ the tensile strength in pounds per square inch. In determining the ductility the elongation shall be measured after breaking on an original length of ten times the shortest dimension of the test piece, in which length must occur the curve of reduction from stretch on both sides of the point of fracture.

Finish and Reduction of Area on Finished Bars.—18. Finished bars must be free from injurious flaws or cracks and must have a workmanlike finish, and round or square test pieces cut therefrom when pulled asunder shall have reduction of area at the point of fracture as above specified.

Number of Test Pieces.—19. For each contract four such tests respectively for reduction of area and for bending, and one additional of each for each 50,000 pounds of steel will, if required, be made by the contractor without charge ; and if the purchaser is not satisfied that the reduction of area test correctly indicates the effect of the heating and rolling, such additional tests for tensile strength, limit of elasticity, and ductility, as he may desire, will be made for him on test pieces conforming to the provisions of clause 3, at the rate of $5.00 each, or, if the contractor desires additional tests, he may make them at his own expense, under the supervision of the purchaser, the quality of the material to be determined by the result of all the tests in the manner set forth in the following clause.

20. Except for tensile strength, the respective requirements

stated are for an average of the tests for each, and the lot of bars or plates from which samples were selected shall be accepted if the tests give such average results; but, if any test piece gives results more than 4 per cent. below said requirements, the particular bar from which it was taken may be rejected, but such tests shall be included in making the average. If any test piece has a manifest flaw, its test shall not be considered. For each bar thus giving results more than 4 per cent. below the requirements, tests from two additional bars shall be furnished by the contractor without charge, and if in a total of not more than ten tests, two bars (or, for a larger number of tests, a proportionately greater number of bars) show results more than 4 per cent. below the requirements, it shall be cause for rejecting the lot from which the sample bars were taken. Such lot shall not exceed 20 tons in weight, and bars of a single pattern, plates rolled in universal mill or in grooves, and sheared plates shall each constitute a separate lot.

Rivet Steel.—21. Rivet steel shall have a specified tensile strength of 60,000 pounds per square inch, and test bars must have a tensile strength within 4,000 pounds per square inch of that specified, and an elastic limit, elongation, and reduction of area at the point of fracture, as stated in clause 17, and be capable of bending double, flat, without sign of fracture on the convex surface of the bend.

Time of Inspection.—22. The inspection and tests of the material will be made promptly on its being rolled, and the quality determined before it leaves the rolling-mill. All necessary facilities for this purpose shall be afforded by the manufacturer; but, if the inspector is not present to make the necessary tests, after due notice given him, then the contractor shall proceed to make such number of tests on the steel then being rolled as may have been agreed upon, or, in the absence of any special agreement, the number provided for in clause 16 or 19, and the quality of such material shall be determined thereby.

Variation of Weights.—23. A variation in cross-section or weight of rolled material of more than $2\frac{1}{2}$ per cent. from that specified may be cause for rejection.

Cast Iron.

24. Except where chilled iron is specified, all castings shall be of tough gray iron free from injurious cold shuts or blow holes, true to pattern, and of a workmanlike finish. Sample pieces 1 inch square cast from the same heat of metal in sand moulds shall be

capable of sustaining on a clear span of 4 feet 6 inches a central load of 500 pounds when tested in the rough bar.

Workmanship.

Inspection.—25. Inspection of the work shall be made as it progresses, and at as early a period as the nature of the work permits.

26. All workmanship must be first-class. All abutting surfaces of compression members, except flanges of plate girders where the joints are fully spliced, must be planed or turned to even bearings so that they shall be in such contact throughout as may be obtained by such means. All finished surfaces must be protected by white lead and tallow.

27. The rivet-holes for splice plates of abutting members shall be so accurately spaced that when the members are brought into position the holes shall be truly opposite before the rivets are driven.

28. When members are connected by bolts which transmit shearing strains the holes must be reamed parallel, and the bolts turned to a driving fit.

29. Rollers must be finished perfectly round and roller-beds planed.

Rivets.—30. Rivets must completely fill the holes, have full heads concentric with the rivet, of a height not less than .6 the diameter of the rivet, and in full contact with the surface, or be countersunk when so required, and machine-driven wherever practicable.

31. Built members must, when finished, be true and free from twists, kinks, buckles, or open joints between the component pieces.

Eye Bars and Pin-hole, and Pilot Nuts.—32. All pin-holes must be accurately bored at right angles to the axis of the members, unless otherwise shown in the drawings, and in pieces not adjustable for length no variation of more than one-thirty-second of an inch will be allowed in the length between centres of pin-holes; the diameter of the pin-holes shall not exceed that of the pins by more than one-thirty-second inch, nor by more than one-fiftieth inch for pins under three and one-half inches diameter. Eye bars must be straight before boring; the holes must be in the centre of the heads, and on the centre line of the bars. Whenever links are to be packed more than one-eighth of an inch to the foot of their length out of parallel with the axis of the structure, they must be bent with a gentle curve until

the head stands at right angles to the pin in their intended position before being bored. All links belonging to the same panel, when placed in a pile, must allow the pin at each end to pass through at the same time without forcing. No welds will be allowed in the body of the bar of eye bars, laterals, or counters, except to form the loops of laterals, counters, and sway rods; eyes of laterals, stirrups, sway rods, and counters, must be bored; pins and lateral bolts must be finished perfectly round and straight, and the party contracting to erect the work must provide pilot nuts where necessary to preserve the threads while the pins are being driven. Thimbles or washers must be used whenever required to fill the vacant spaces on pins or bolts.

Tests of Eyes on Full Size Bars.—33. To determine the strength of the eyes, full size eye bars or rods with eyes may be tested to destruction, provided notice is given in advance of the number and size required for this purpose, so that the material can be rolled at the same time as that required for the structure, and any lot of iron bars from which full size samples are tested shall be accepted—

1st, if not more than one-third the bars tested break in the eye; or,

2d, if more than one-third do break in the eye and the average of the tests of those which so break shows a tensile strength in pounds per square inch of original bar, given by the formula—

$$52{,}000 - \frac{7{,}000 \times \text{area of original bar}}{\text{circumference of original bar}} - 500 \times \text{width of bar (all in}$$

inches), and not more than one-half of those which break in the eye fail at more than 5 per cent. below the strength given by the formula. Any lot of steel bars from which full size samples are tested shall be accepted if the average of the tests shows a strength per square inch of original bar, in those which break in the eye, within 4,000 pounds of that specified, as in clause 17; but if one-half the full size samples break in the eye, it shall be cause for rejecting the lot from which the sample bars were taken. All full size sample bars which break in the eye at less than the strength here specified shall be at the expense of the contractor, unless he shall have made objection in writing to the form or dimension of the heads before making the eye bars. All others shall be at the expense of the purchaser. If the contractor desires additional tests they shall be made at his own expense, under the supervision of the purchaser, the acceptance of the bars to be determined by the result of all the tests in the manner above set forth. A variation from the specified dimensions

of the heads will be allowed, in thickness of one-thirty-second inch below and one-sixteenth above that specified, and in diameter of one-fourth inch in either direction.

Punching and Reaming.—34. In iron work, the diameter of the punch shall not exceed by more than one-sixteenth inch the diameter of the rivets to be used. Rivet-holes must be accurately spaced ; the use of drift-pins will be allowed only for bringing together the several parts forming a member, and they must not be driven with such force as to disturb the metal about the holes ; if the hole must be enlarged to admit the rivet, it must be remade; all rivet-holes in steel work, if punched, shall be made with a punch one-eighth inch in diameter less than the diameter of the rivet intended to be used, and shall be reamed to a diameter one-sixteenth inch greater than the rivet.

Annealing.—35. In all cases where a steel piece in which the full strength is required has been partially heated, the whole piece must be subsequently annealed. All bends in steel must be made cold, or if the degree of curvature is so great as to require heating, the whole piece must be subsequently annealed.

Painting.—36. All surfaces inaccessible after assembling must be well painted or oiled before the parts are assembled.

37. The decision of the engineer shall control as to the interpretation of drawings and specifications during the execution of work thereunder, but this shall not deprive the contractor of his right to redress, after the completion of the work, for an improper decision.

TABLE II.

Tables showing the Strength given by the Formulæ of Sections 4, 6, and 33, for Iron Bars of Various Dimensions.

For Standard Test Piece of Bars, $52{,}000 - \dfrac{7{,}000 \times \text{area of original bar}}{\text{circumference of original bar}}$.

For eyes of Full Size Eye Bars,

$$52{,}000 - \frac{7{,}000 \times \text{area of original bar}}{\text{circumference of original bar}} - 500 \text{ lbs. per inch of width.}$$

For Standard Test Piece of Angles, $50{,}000 - \dfrac{7{,}000 \times \text{area of original bar}}{\text{circumference of original bar}}$.

Size of bar.	Standard test piece.	Eyes of full size eye bars.	Size of angle.	Standard test piece.
1 × 1	50,250	49,750	6 × 6 × ½	48,320
1¼ × 1¼	49,820	49,195	6 × 6 × ⅞	47,165
1½ × 1½	49,380	48,630		
2 × 2	48,500	47,500	4 × 4 × ⅜	48,750
2 × ½	50,600	49,600	4 × 4 × ¾	47,620
2 × ¾	50,090	49,090	3 × 3 × ¼	49,160
2 × 1	49,670	48,670	3 × 3 × $\frac{11}{16}$	47,870
3 × ½	50,510	49,010		
3 × ¾	49,900	48,400	2 × 2 × ¼	49,180
3 × 1	49,380	47,880	2 × 2 × ⅜	48,810
4 × ¾	49,790	47,790		
4 × 1	49,200	47,200		
4 × 1¼	48,670	46,670		
5 × ¾	49,720	47,220		
5 × 1	49,090	46,590		
5 × 1¼	48,500	46,000		
5 × 1½	47,960	45,460		
5 × 2	47,010	44,510		
6 × ¾	49,670	46,670		
6 × 1	49,000	46,000		
6 × 1¼	48,390	45,390		
6 × 1½	47,800	44,800		
6 × 2	46,750	43,750		
7 × 1	48,940	45,440		
7 × 1¼	47,680	44,180		
7 × 2	46,560	43,060		

TABLE III.

Strength of Iron Rods.[1]

SAFE TENSILE STRENGTHS OF ROUND WROUGHT-IRON RODS ½ TO 4 INCHES IN DIAMETER, AND THE WEIGHTS PER FOOT, THE SAFE STRENGTH BEING TAKEN AT 10,000 POUNDS PER SQUARE INCH.

Tensile Strength and Quality of Wrought-Iron.

The best American rolled iron has a breaking tensile strength of from fifty thousand to sixty thousand pounds per square inch for specimens not exceeding one square inch in section. Ordinary bar-iron should not break under a less strain than fifty thousand pounds per square inch, and should not take a *set* under a stress less than twenty-five thousand pounds per square inch. A bar one inch square and one foot long should stretch fifteen per cent of its length before breaking, and should be capable of being bent, cold, 90° over the edge of an anvil without sign of fracture, and should show a fibrous texture when broken.

Iron that will not meet these requirements is not suitable for structures; but nothing is gained by specifying more severe tests, because, in bars of the sizes and shapes usually required for such work, nothing more can be attained with certainty, and conscientious makers will be unwilling to agree to furnish that which it is not practicable to produce.

The *working-strength* of wrought-iron ties in trusses is generally

[1] For steel, increase by 20 per cent.

taken at ten thousand pounds per square inch. In places where the load is perfectly steady and constant, twelve thousand pounds may be used.

The *extension of iron*, for all practical purposes, is as follows :—

Wrought-iron, $\frac{1}{10000}$ of its length per ton per square inch.
Cast-iron, $\frac{1}{5000}$ of its length per ton per square inch.

Appearance of the Fractured Surface of Wrought-Iron.

At one time it was thought that a fibrous fracture was a sign of good tough wrought-iron, and that a crystalline fracture showed that the iron was bad, hard, and brittle. Mr. Kirkaldy's experiments, however, show conclusively, that, whenever wrought-iron breaks suddenly, it invariably presents a crystalline appearance; and, when it breaks gradually, it invariably presents a fibrous appearance. From the same experiments it was also shown, that the appearance of the fractured surface of wrought-iron is, to a certain extent, an indication of its quality, provided it is known how the stress was applied which produced the fracture.

Small, uniform crystals, of a uniform size and color, or fine, close, silky fibres, indicate a good iron.

Coarse crystals, blotches of color caused by impurities, loose and open fibres, are signs of bad iron; and flaws in the fractured surface indicate that the piling and welding processes have been imperfectly carried out.

Kirkaldy's Conclusions.[1]

Mr. David Kirkaldy of England, who made some of the most valuable experiments on record, on the strength of wrought-iron, came to some conclusions, many of which differed from what had previously been supposed to be true.

The following are of special importance to the student of building construction, and should be carefully studied: —

"The breaking-strain does *not* indicate the quality, as hitherto assumed.

"A *high* breaking-strain may be due to the iron being of superior quality, dense, fine, and moderately soft, or simply to its being very hard and unyielding.

"A *low* breaking-strain may be due to looseness and coarseness in the texture; or to extreme softness, although very close and fine in quality.

---

[1] Kirkaldy's Experiments on Wrought-Iron and Steel.

"The contraction of area at fracture, previously overlooked, fo an essential element in estimating the quality of specimens.

"The respective merits of various specimens can be correctly as tained by comparing the breaking-strain *jointly* with the contrac of area.

"Inferior qualities show a much greater variation in the breaki strain than superior.

"Greater differences exist between small and large bars in co than in fine varieties.

"The prevailing opinion of a rough bar being stronger tha turned one is erroneous.

"Rolled bars are slightly hardened by being forged down.

"The breaking-strain and contraction of area of iron plates greater in the direction in which they are rolled than in a tra verse direction.

"Iron is less liable to snap, the more it is worked and rolled.

"The ratio of ultimate elongation may be greater in short tl in long bars, in some descriptions of iron; whilst in others the ra is not affected by difference in the length.

"Iron, like steel, is softened, and the breaking-strain reduced, being heated, and allowed to cool slowly.

"A great variation exists in the strength of iron bars which h been cut and welded. Whilst some bear almost as much as uncut bar, the strength of others is reduced fully a third.

"The welding of steel bars, owing to their being so easily bur by slightly overheating, is a difficult and uncertain operation.

"Iron is injured by being brought to a white or welding heat not at the same time hammered or rolled.

"The breaking-strain is considerably less when the strain is appl suddenly instead of gradually, though some have imagined that reverse is the case.

"The specific gravity is found generally to indicate pretty corre the quality of specimens.

"The density of iron is *decreased* by the process of wire-draw and by the similar process of cold rolling,[1] instead of *increased*, previously imagined.

"The density of iron is decreased by being drawn out unde tensile strain, instead of increased, as believed by some.

"It must be abundantly evident, from the facts which have b

---

[1] The conclusion of Mr. Kirkaldy in respect to cold rolling is undoubtedly t when the rolling amounts to wire-drawing; but, when the compression of surface by rolling diminishes the sectional area in greater proportion tha extends the bar, the result, according to the experience of the Pittsburgh ma facturers, is a slight increase in the density of the iron.

produced, that the *breaking-strain*, when taken alone, gives a false impression of, instead of indicating, the real quality of the iron, as the experiments which have been instituted reveal the somewhat *startling* fact, that frequently the inferior kinds of iron actually yield a higher result than the superior. The *reason* of this difference was shown to be due to the fact, that, whilst the one quality retained its original area only very slightly decreased by the strain, the *other* was reduced to less than one-half. Now, surely this variation, hitherto unaccountably *completely overlooked*, is of importance as indicating the relative hardness or softness of the material, and thus, it is submitted, forms an essential element in considering the *safe load* that can be practically applied in various structures. *It must be borne in mind*, that, although the softness of the material has the effect of lessening the amount of the breaking-strain, it has the very opposite effect as regards the *working-strain.* This holds good for two reasons: first, the softer the iron, the less liable it is to snap; and, second, fine or soft iron, being more uniform in quality, can be more *depended upon in practice.* Hence the load which this description of iron can suspend with safety may approach much more nearly the limit of its breaking-strain than can be attempted with the harder or coarser sorts, where a greater margin must necessarily be left.

"As a necessary corollary to what we have just endeavored to establish, the writer now submits, in addition, that the *working-strain* should be in proportion to the breaking-strain per square inch of *fractured area*, and not to the breaking-strain per square inch of *original area*, as heretofore. Some kinds of iron experimented on by the writer will sustain with safety more than double the load that others can suspend, especially in circumstances where the load is unsteady, and the structure exposed to concussions, as in a ship or railway bridge."

Eye-Bars and Screw-Ends.

Iron ties are generally of flat or round bars attached by eyes and pins, or by screw-ends. In either case, it is essential that the proportion of the eyes or screw-ends shall be such that the tie will not break at the end sooner than in the middle. In important work, eyes are forged on the ends of flat or round bars, by hydraulic pressure, in suitably shaped dies; and, while the risk of a welded eye is thus avoided, a solid and well-formed eye is made from the iron of the bar itself.

A similar process is adopted for enlarging the screw-ends of long

rods ; so that, when the screw is cut, the diameter of the screw at the root of the thread is left *a little larger than the body of the rod.* Frequent trials with such rods has proven that they will pull apart in tension anywhere else but in the screw ; the threads remaining perfect, and the nut turning freely after having been subjected to such a severe test. By this means the net section required in tension is made available with the least excess of material, and no more dead weight is put upon the structure than is actually needed to carry the loads imposed.

The diameter of the eye in flat bars, having the same thickness throughout, should be 0.8 the width of the bar. The width of the metal on each side of the eye should be $\frac{3}{4}$ the width of the bar, and in front of the eye should be equal to the width of the bar. When it becomes necessary to use a larger pin than here described (as when a bar takes hold of the same pin with bars of larger size), the amount of metal around the eye should be still further increased. The weight of an eye-bar, proportioned as here described, will be about equal to that of a plain bar of a length equal to the distance from centre to centre of the pins, plus twice the diameter of the pin multiplied by the width of bar, both in inches.

The thickness of flat bars should be at least *one-fourth of the width* in order to secure a good bearing surface on the pin, and the metal at the eyes should be as thick as the bars on which they are upset.

Table IV. gives the proportion for eye-bars, sleeve nuts, and clevises, as manufactured by the *New Jersey Steel & Iron Co.*

Table VI. gives the proportion for upset screw-ends for different sizes of rods, as adopted by the *Keystone Bridge Company.*

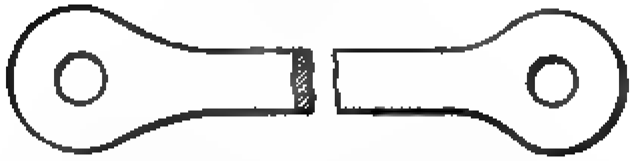
Cast-Iron has only about one-third the tensile strength of wrought-iron; and as it is liable to air-holes, internal strains from unequal contraction in cooling, and other concealed defects, reducing its effective area for tension, it should never be used where it is subject to any great tensile stress.

Tables.

The following tables give the strength of iron rods, bars, steel and iron wire ropes, manila ropes, and dimensions of upset screw-ends.

The diameter in Table III. is the least diameter of the rod; and, if the screw is cut into the rod without enlarging the end, the effective diameter between the threads of the screw should be used in calculating the strength of the rod.

TABLE IV.



WELDLESS, DIE-FORGED EYE BARS,

As Manufactured by the New Jersey Steel and Iron Co.

Width of bar.	Width of head.	Diam. of pin.*
7 inch.	16[illegible] inch.	5[illegible] inch
6 inch.	15 "	5[illegible] "
"	14[illegible] "	5[illegible] "
"	14 "	4[illegible] "
5 inch.	13 "	5[illegible] "
"	12[illegible] "	4[illegible] "
"	12 "	4[illegible] "
"	11[illegible] "	3[illegible] "
4½ inch.	11[illegible] "	4[illegible] "
"	10[illegible] "	3[illegible] "
"	10[illegible] "	3[illegible] "
4 inch.	11[illegible] "	4[illegible] "
"	10[illegible] "	4[illegible] "
"	10 "	3[illegible] "
"	9[illegible] "	3[illegible] "
"	9[illegible] "	3[illegible] "
3½ inch	10[illegible] "	4[illegible] "
"	10 "	4[illegible] "
"	9[illegible] "	3[illegible] "
"	8[illegible] "	3[illegible] "
"	8[illegible] "	2[illegible] "
3 inch.	10 "	4[illegible] "
"	9[illegible] "	4[illegible] "
"	9 "	3[illegible] "
"	8[illegible] "	3[illegible] "
"	7[illegible] "	2[illegible] "
"	7 "	2[illegible] "
2½ inch.	6[illegible] "	2[illegible] "
"	5[illegible] "	1[illegible] "
2 inch	7 "	2[illegible] "
"	6 "	2[illegible] "
"	6[illegible] "	1[illegible] "

* The smallest diameter of pin given for each width of bar is the standard size. The larger sizes given are the largest that are allowable with each head.

The thickness of the bars should not be more than ½ nor less than ¼ their width. Eye-bars are bored $\frac{1}{32}$ inch larger than the diameter of the pin. Other sizes can be furnished.

Table VII. was compiled from data furnished by the John A. Roebling's Sons Company of New York.

The ropes with nineteen wires to the strand are the most pliable, and are generally used for hoisting and running rope. The ropes with seven wires to the strand are stiffer, and are better adapted for standing rope, guys, and rigging.

Table IX. is taken from Trautwine's "Pocket-Book for Engineers."

Table X. gives the weight and proof, or safe strength, of chains manufactured by the New Jersey Steel and Iron Company.

TABLE V.

Safe Strength of Flat Rolled Iron Bars.

TE.—Computed at 10,000 lbs. per square inch.

TABLE V. (*concluded*).

Safe Strength of Flat Rolled Iron Bars.

Thickness in inches.	Width in inches.									
	$3\frac{1}{2}''$	$3\frac{3}{4}''$	$4''$	$4\frac{1}{4}''$	$4\frac{1}{2}''$	$4\frac{3}{4}''$	$5''$	$5\frac{1}{2}''$	$6''$	$6\frac{1}{2}''$
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
$\frac{1}{16}$	2,190	2,340	2,500	2,660	2,810	2,970	3,130	3,440	3,750	4,060
$\frac{1}{8}$	4,380	4,690	5,000	5,310	5,630	5,940	6,250	6,880	7,500	8,130
$\frac{3}{16}$	6,560	7,030	7,500	7,970	8,440	8,910	9,380	10,300	11,300	12,200
$\frac{1}{4}$	8,750	9,380	10,000	10,600	11,300	11,900	12,500	13,800	15,000	16,300
$\frac{5}{16}$	10,900	11,700	12,500	13,300	14,100	14,800	15,600	17,200	18,800	20,300
$\frac{3}{8}$	13,100	14,100	15,000	15,900	16,900	17,800	18,800	20,600	22,500	24,400
$\frac{7}{16}$	15,300	16,400	17,500	18,600	19,700	20,800	21,900	24,100	26,300	28,400
$\frac{1}{2}$	17,500	18,800	20,000	21,300	22,500	23,800	25,000	27,500	30,000	32,500
$\frac{9}{16}$	19,700	21,100	22,500	23,900	25,300	26,700	28,100	30,900	33,800	36,600
$\frac{5}{8}$	21,900	23,400	25,000	26,600	28,100	29,700	31,300	34,400	37,500	40,600
$\frac{11}{16}$	24,100	25,800	27,500	29,200	30,900	32,700	34,400	37,800	41,300	44,700
$\frac{3}{4}$	26,300	28,100	30,000	31,900	33,800	35,600	37,500	41,300	45,000	48,800
$\frac{13}{16}$	28,400	30,500	32,500	34,500	36,600	38,600	40,600	44,700	48,800	52,800
$\frac{7}{8}$	30,600	32,800	35,000	37,200	39,400	41,600	43,800	48,100	52,500	56,900
$\frac{15}{16}$	32,800	35,200	37,500	39,800	42,200	44,500	46,900	51,600	56,300	60,900
1	35,000	37,500	40,000	42,500	45,000	47,500	50,000	55,000	60,000	65,000
$1\frac{1}{16}$	37,200	39,800	42,500	45,200	47,800	50,500	53,100	58,400	63,800	69,100
$1\frac{1}{8}$	39,400	42,200	45,000	47,800	50,600	53,400	56,300	61,900	67,500	73,100
$1\frac{3}{16}$	41,600	44,500	47,500	50,500	53,400	56,400	59,400	65,300	71,300	77,200
$1\frac{1}{4}$	43,800	46,900	50,000	53,100	56,300	59,400	62,500	68,800	75,000	81,300
$1\frac{3}{8}$	48,100	51,600	55,000	58,400	61,900	65,300	68,800	75,600	82,500	89,400
$1\frac{1}{2}$	52,500	56,300	60,000	63,800	67,500	71,300	75,000	82,500	90,000	97,500
$1\frac{5}{8}$	56,900	60,900	65,000	69,100	73,100	77,200	81,300	89,400	97,500	105,600
$1\frac{3}{4}$	61,300	65,600	70,000	74,400	78,800	83,100	87,500	96,300	105,000	113,800
$1\frac{7}{8}$	65,600	70,300	75,000	79,700	84,400	89,100	93,800	103,100	112,500	121,900
2	70,000	75,000	80,000	85,000	90,000	95,000	100,000	110,000	120,000	130,000

TABLE VI.

Upset Screw-Ends for Round and Square Bars.

STANDARD PROPORTIONS OF THE KEYSTONE BRIDGE COMPANY.

TABLE VI. (*concluded*).

Upset Screw-Ends.

TABLE VII.

Strength of Iron and Steel Wire Ropes,

MANUFACTURED BY THE JOHN A. ROEBLING'S SONS CO., NEW YORK.

Trade number.	Diameter in inches.	Weight per foot in lbs. of rope with	Iron.		Cast-Steel.	
			Breaking	Proper working	Breaking	Proper working

Ropes, Hawsers, and Cables.

(HASWELL.)

Ropes of hemp fibres are laid with three or four strands of twisted fibres, and run up to a circumference of twelve inches.

Hawsers are laid with three strands of rope, or with four rope strands.

Cables are laid with three strands of rope only.

Tarred ropes, hawsers, etc., have twenty-five per cent less strength than white ropes: this is in consequence of the injury the fibres receive from the high temperature of the tar, — 290°.

Tarred hemp and manila ropes are of about equal strength. Manila ropes have from twenty-five to thirty per cent less strength than white ropes. Hawsers and cables, from having a less proportionate number of fibres, and from the increased irregularity of the resistance of the fibres, have less strength than ropes; the difference varying from thirty-five to forty-five per cent, being greatest with the least circumference.

Ropes of four strands, up to eight inches, are fully sixteen per cent stronger than those having but three strands.

Hawsers and cables of three strands, up to twelve inches, are fully ten per cent stronger than those having four strands.

The absorption of tar in weight by the several ropes is as follows: —

Bolt-rope	18 per cent	Cables	21 per cent
Shrouding	15 to 18 per cent	Spun-yarn	25 to 30 per cent

White ropes are more durable than tarred.

The greater the degree of twisting given to the fibres of a rope, etc., the less its strength, as the exterior alone resists the greater portion of the strain.

To compute the Strain that can be borne with Safety by New Ropes, Hawsers, and Cables, deduced from the Experiments of the Russian Government upon the Relative Strength of Different Circumferences of Ropes, Hawsers, etc.

The United-States navy test is 4200 pounds for a white rope, of three strands of best Riga hemp, of one and three-fourths inches in circumference (i.e., 17,000 pounds per square inch); but in the following table 14,000 pounds is taken as the unit of strain that can be borne with safety.

RULE. — Square the circumference of the rope, hawser, etc., and multiply it by the following units for ordinary ropes, etc.

TABLE VIII.

Showing the Units for computing the Safe Strain that may be borne by Ropes, Hawsers, and Cables.

When it is required to ascertain the weight or strain that can be borne by ropes, etc., in general use, the above units should be reduced *one-third*, in order to meet the reduction of their strength by chafing, and exposure to the weather.

TABLE IX.

Strength and Weight of Manila Rope.

TABLE X.

Weight and Proof Strength of Chain.

MANUFACTURED BY THE NEW-JERSEY STEEL AND IRON COMPANY.

Strength of Old Iron.—A square link 12 inches broad, 1 inch thick, and about 12 feet long was taken from the Kieff Bridge, then 40 years old, and tested in comparison with a similar link which had been preserved in the stock-house since the bridge was built. The following is a record of a mean of four longitudinal test pieces, $1 \times 1\frac{1}{8} \times 8$ inches, taken from each link.

	Old link from bridge.	New link from storehouse.
Tensile strength per square inch, tons	21.8	22.2
Elastic limit per square inch, tons	11.1	11.9
Elongation, per cent	14.05	18.42
Contraction, per cent	17.85	18.75

(The Mechanical World, London.)

CHAPTER X.

RESISTANCE TO SHEARING.

By shearing is meant the pushing of one part of a piece by the other. Thus in Fig. 1, let *abcd* be a beam resting upon the supports *SS*, which are very near together. If a sufficiently heavy

Fig. 1.

load were placed upon the beam, it would cause the beam to break, not by bending, but by pushing the whole central part of the beam through between the ends, as represented in the figure. This mode of fracture is called "shearing."

The resistance of a body to shearing is, like its resistance to tension, directly proportional to the area to be sheared. Hence, if we denote the resistance of one square inch of the material to shearing by F, we shall have as the safe resistance to shearing,

$$\left.\begin{array}{c}\text{Safe shearing}\\ \text{strength}\end{array}\right\} = \frac{\text{area to be sheared} \times F}{S}, \qquad (1)$$

S denoting factor of safety, as before.

A piece of timber may be sheared either longitudinally or transversely; and, as the resistance is not the same in both cases, the value of F will be different in the two cases. Hence, in substituting values for F, we must distinguish whether the force tends to shear the piece longitudinally (lengthwise), or transversely (across).

Table I. gives the values of F, as determined by experiment, for the most common materials employed in architectural con uction.

TABLE I.

Showing the Resistance of Materials to Shearing, both Longitudinally and Transversely, or the Values of F.

Materials	Values of *F*.	
	Longitudinally.	Transversely.
	lbs.	lbs.
Cast-iron	–	27,700 [a]
Wrought iron	–	50,000 [a]
Steel	–	63,746 [b]
White ash	–	1,400 [a]
Beech	–	5,200 [c]
Birch	–	5,600 [c]
Hemlock	540 [d]	2,700 [c]
Locust	1,180 [d]	7,000 [c]
White oak	780 [d]	4,400 [c]
White pine	490 [d]	2,480 [c]
Yellow pine	510 [d]	5,700 [c]
Spruce	470 [d]	3,256 [c]
Black walnut	–	2,000 [a]
Oregon pine	840 [e]	–
Oregon spruce	310 [e]	–
Oregon ash	784 [e]	–
Oregon maple	732 [e]	–

There are but few cases in architectural construction in which the resistance to shearing has to be provided for. The one most frequently met with is at the end of a tie-beam, as in Fig. 2.

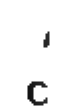


Fig. 2.

The rafter *R* exerts a thrust which tends to push or shear off the piece *ABCD*, and the area of the section at *CD* should offer enough resistance to keep the rafter in place. This area is equal to *CD*

[a] Rankine. [b] Kirkaldy. [c] Trautwine. [d] Hatfield. [e] United-States Government at Watertown Arsenal.

times the breadth of the tie-beam; and, as the breadth is fixed, we have to determine the length, CD. If we let H denote the horizontal thrust of the rafter, then, by a simple deduction from formula 1, we have the rule:—

$$\text{Length of } CD \text{ in inches} = \frac{S \times H}{\text{breadth of beam} \times F}, \qquad (2)$$

F, in this case, being the resistance to shearing longitudinally.

EXAMPLE I. — The horizontal thrust of a rafter is 20,000 pounds, the tie-beam is of Oregon pine, and is ten inches wide: how far should the beam extend beyond the point D?

Ans. In this case $H = 20{,}000$ pounds, and from Table I. we find that $F = 840$; S we will take at 5. Then

$$CD = \frac{5 \times 20000}{10 \times 840}, \text{ or nearly 12 inches.}$$

Practically a large part of the thrust is generally taken up by an iron bolt or strap passed through or over the foot of the rafter and tie-beam, as at A (Fig. 2). When this is done, the rod or strap should be as obliquely inclined to the beam as is possible; and, whenever it can be done, a strap should be used in preference to a rod, as the rod cuts into the wood, and thus weakens it.

The two principal cases in building construction where the shearing strength must be computed, are pins and rivets; for the latter see pages 557–565.

Strength of Pins in Iron Bridge and Roof Trusses. —Iron and steel trusses are now so generally used that it is necessary for the architect who is at all advanced in his profession to know how to determine the strength of the joints, and especially of pin joints ; and to facilitate the calculation of the necessary size of pins, we give Table II , which shows the single shearing strength and bearing value of pins, and Table III., showing the maximum bending moment allowed in pins.

Pins must be calculated for shearing, bending, and bearing strains, but one of the latter two only (in almost every case) determines the size to be used.

By bearing strain is meant the force required to crush the edges of the iron plates against which the pin bears.

The several strains usually allowed per square inch on pin connections in bridges are : shearing, 7,500 pounds ; crushing, 12,000 pounds ; and bending, 15,000 pounds for iron, and 20,000 pounds for steel.

The shearing strain is measured on the area of cross-section ; the

crushing strain, on the area measured by the product of the diameter of the pin, by the thickness of the plate or web on which it bears.

The bending moment is determined by the same rules as given for determining the bending moment of beams.

When groups of bars are connected to the same pin, as in the lower chords of trusses, the sizes of bars must be so chosen, and the bars so placed, that at no point on the pin will there be an excessive bending strain, on the presumption that all the bars are strained equally per square inch.

The following example will show the method of determining the size of pin in a simple joint.

EXAMPLE.—Fig. 3. Determine the size of pin for the joint in the lower chord of a truss, shown in Fig. 3, the middle bar being a vertical suspension rod, merely to hold the chord in place.

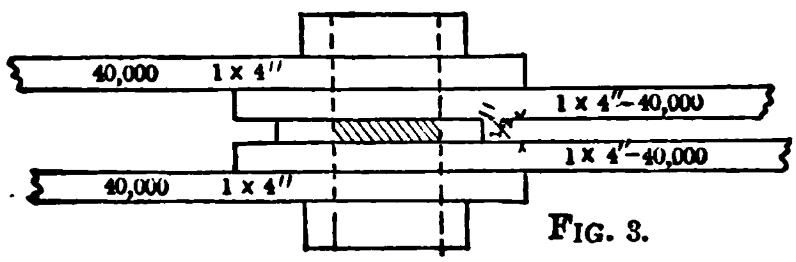


FIG. 3.

Ans. The shearing and crushing strain in this case is 40,000 pounds. The bending moment will be 40,000 × 1″; the distance between the centres of the two outer bars = 40,000 pounds. From Table III., we find that to sustain a bending moment of 40,000 lbs., with a fibre strain of 15,000 lbs., will require a 3″ or $3\frac{1}{8}$″ pin. From Table II., we find that the bearing value of a $3\frac{1}{8}$″ pin is but 37,500 lbs., and that we must increase the size of the pin to $3\frac{3}{8}$ inches. The shearing strength of a $3\frac{3}{8}$″ pin is, from Table II., 67,500 lbs., so that the size of pin we must use in this case is determined by the bearing strain. To be sure of the correct size of the pin, one must make the calculation for all three of the strains.

TABLE II.

Shearing and Bearing Values of Pins for One Inch Thickness of Plate.

Diameter of pin.	Area of pin.	Bearing value at 12,000 lbs. per sq. in.	Single shear 7,500 lbs. per sq. in.	Diameter of pin.	Area of pin.	Bearing value at 12,000 lbs. per sq. in.	Single shear 7,500 lbs. per sq. in.	Diameter of pin.	Area of pin.	Bearing value at 12,000 lbs. per sq. in.	Single shear 7,500 lbs. per sq. in.
Inches.	Sq. in.	Lbs.	Lbs.	Inches.	Sq. in.	Lbs.	Tons.	Inches.	Sq. in.	Lbs.	Tons.
1	.785	12,000	5,890	3	7.069	36,000	26.5	5	19.64	60,000	73.6
$1\frac{1}{8}$	.994	13,500	7,455	$3\frac{1}{8}$	7.670	37,500	28.7	$5\frac{1}{8}$	20.63	61,500	77.3
$1\frac{1}{4}$	1.227	15,000	9,202	$3\frac{1}{4}$	8.296	39,000	31.0	$5\frac{1}{4}$	21.65	63,000	81.2
$1\frac{3}{8}$	1.485	16,500	11,132	$3\frac{3}{8}$	8.946	40,500	33.5	$5\frac{3}{8}$	22.69	64,500	85.1
$1\frac{1}{2}$	1.767	18,000	13,252	$3\frac{1}{2}$	9.621	42,000	36.0	$5\frac{1}{2}$	23.76	66,000	89.1
$1\frac{5}{8}$	2.074	19,500	15,555	$3\frac{5}{8}$	10.32	43,500	38.7	$5\frac{5}{8}$	24.85	67,500	93.2
$1\frac{3}{4}$	2.405	21,000	18,037	$3\frac{3}{4}$	11.05	45,000	41.4	$5\frac{3}{4}$	25.97	69,000	97.3
$1\frac{7}{8}$	2.761	22,500	20,707	$3\frac{7}{8}$	11.79	46,500	44.2	$5\frac{7}{8}$	27.11	70,500	101.1
2	3.142	24,000	23,565	4	12.57	48,000	47.0	6	28.27	72,000	106
$2\frac{1}{8}$	3.547	25,500	26,600	$4\frac{1}{8}$	13.36	49,500	50.1	$6\frac{1}{8}$	29.46	73,500	110
$2\frac{1}{4}$	3.976	27,000	29,820	$4\frac{1}{4}$	14.19	51,000	53.2	$6\frac{1}{4}$	30.68	75,000	115
$2\frac{3}{8}$	4.430	28,500	33,225	$4\frac{3}{8}$	15.03	52,500	56.3	$6\frac{3}{8}$	31.92	76,500	119
$2\frac{1}{2}$	4.909	30,000	36,817	$4\frac{1}{2}$	15.90	54,000	59.6	$6\frac{1}{2}$	33.18	78,000	124
$2\frac{5}{8}$	5.412	31,500	40,590	$4\frac{5}{8}$	16.80	55,500	63.0	$6\frac{5}{8}$	34.47	79,500	129
$2\frac{3}{4}$	5.940	33,000	44,550	$4\frac{3}{4}$	17.72	57,000	66.3	$6\frac{3}{4}$	35.79	81,000	134
$2\frac{7}{8}$	6.492	34,500	48,690	$4\frac{7}{8}$	18.67	58,500	70.0	$6\frac{7}{8}$	37.12	82,500	139

TABLE III.

Maximum Bending Moments to be Allowed on Pins for Maximum Fibre Strains of 15,000, 20,000, *and* 22,500 *Pounds per Square Inch.**

Diameter of pin.	Moment for $S=15{,}000$.	Moment for $S=20{,}000$.	Moment for $S=22{,}500$.	Diameter of pin.	Moment for $S=15{,}000$.	Moment for $S=20{,}000$.	Moment for $S=22{,}500$.
Inches.	Lbs. in.	Lbs. in.	Lbs. in.	Inches.	Lbs. in.	Lbs. in.	Lbs. in.
1	1,470	1,960	2,210	4	94,200	125,700	141,400
1⅛	2,100	2,800	3,140	4⅛	103,400	137,800	155,000
1¼	2,830	3,830	4,310	4¼	113,000	150,700	169,600
1⅜	3,830	5,100	5,740	4⅜	123,300	164,400	185,000
1½	4,970	6,630	7,460	4½	134,200	178,900	201,300
1⅝	6,320	8,430	9,480	4⅝	145,700	194,300	218,500
1¾	7,890	10,500	11,800	4¾	157,800	210,400	236,700
1⅞	9,710	12,900	14,600	4⅞	170,600	227,500	255,900
2	11,800	15,700	17,700	5	184,100	245,400	276,100
2⅛	14,100	18,800	21,200	5⅛	198,200	264,300	297,300
2¼	16,800	22,400	25,200	5¼	213,100	284,100	319,600
2⅜	19,700	26,300	29,600	5⅜	228,700	304,900	343,000
2½	23,000	30,700	34,500	5½	245,000	326,700	367,500
2⅝	26,600	35,500	40,000	5⅝	262,100	349,500	393,100
2¾	30,600	40,800	45,900	5¾	280,000	373,300	419,900
2⅞	35,000	46,700	52,500	5⅞	298,600	398,200	447,900
3	39.800	53,000	59,600	6	318,100	424,100	477,100
3⅛	44,900	59,900	67,400	6⅛	338.400	451,200	507,600
3¼	50,600	67,400	75,800	6¼	359,500	479,400	539,300
3⅜	56,600	75,500	84,900	6⅜	381,500	508,700	572,300
3½	63,100	84,200	94,700	6½	404,400	539,200	606,600
3⅝	70,100	93,500	105,200	6⅝	428,200	570,900	642,300
3¾	77,700	103,500	116,500	6¾	452,900	603,900	679,400
3⅞	85,700	114,200	128,500	6⅞	478,500	638,000	717,800

Remarks—The following is the formula for flexure applied to pins:

$$M = \frac{S \pi d^3}{32} \quad \text{or} \quad = \frac{S A d}{8}$$

M = moment of forces for any section through pin.

S = strain per sq. in. in extreme fibres of pin at that section.

A = area of section.

d = diameter.

$\pi = 3.14159$.

The forces are assumed to act in a plane passing through the axis of the pin.

The above table gives the values of M for different diameters of pin, and for three values of S.

If M max. is known, an inspection of the table will therefore show what diameter of pin must be used in order that S may not exceed 15,000, 20,000, or 22,500 lbs., as the requirements of the case may be.

For Railroad Bridges proportioned to a factor of safety of 5, it is customary to make S max. = 15,000 lbs. in iron and = 20,000 lbs. in steel.

* Carnegie, Phipps & Co.'s Hand-book.

Bending Moment in Pins.

The only difficult part of the process of calculating the sizes of pins will generally be found in determining the bending moment. In cases where the strains all act in the same plane, the bending moment can generally be determined by multiplying the outside force by the distance from its centre to the centre of the next bar, as in the foregoing example. When, however, the forces act in several planes, as is generally the case, the process of determining the bending moment is more difficult, and can be best determined by a graphic process, first published by Prof. Chase Green, and included in his lectures to the students in engineering at the University of Michigan.

As the pieces acting on any well-designed joint are symmetrically arranged, it is unnecessary to consider more than one-half of their number. Fig. 4 shows a sketch of one-half the members of a joint in the lower chord of a Howe truss. The pieces are parallel to the plane of the paper, and the pin is perpendicular to the same, but drawn in cabinet perspective, at an angle of 45° with a horizontal.

The bars are assumed to be each one inch thick, and the channel to have one-half-inch web. The centre of the hanger is ¾" from the centre of the channel.

The method of obtaining the bending moment is as follows :

Draw the line *A B* at an angle of 45° with a horizontal, and, commencing with *c*, lay off the distances between the centres of the bars to a scale (1½" or 3" to the foot will be found most convenient) ; then draw the lines 1–2, 2–3, etc., parallel to the pieces which they represent in the truss, to a scale of pounds. Resolve the oblique forces into their horizontal and vertical components (in this example there is but one oblique force).

Next draw the stress diagram (Fig. 6) as follows: On a horizontal line lay off 1–2 equal to the first or outer force ; 2–3, equal to the next, 3–4 ; and 4–1, being the horizontal component of the brace, closes the figure. In the same way, lay off the vertical forces 1 5, 5 6, 6 1. If the forces are correct, the sum of the forces acting in one direction will always equal those acting in the opposite direction. From 1 draw the line 1 0 at 45°, equal to the same scale of, say, 20,000 pounds, or any other convenient length. Draw 0 2, 0 3, 0 4, etc. Then, in Fig. 5, starting at the first horizontal force, draw *c d* parallel to 0 2, *d e* parallel to 0 3, *e f* parallel to 0 4, and *f k* parallel to 0 1.

In the same way, starting at the first vertical force, draw *r s* parallel to 0 5, *s t* parallel to 0 6, and *t* 2 parallel to 0

line $c\ d\ e\ f\ k$ will represent the boundary of the horizontal ordinates, and $r\ s\ t\ v$ the boundary of the vertical ordinates. And to find the resultant of these ordinates at any point on the pin, it is

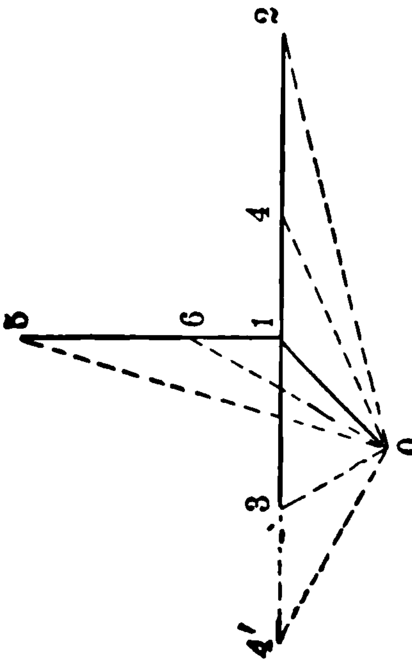


FIG. 6.

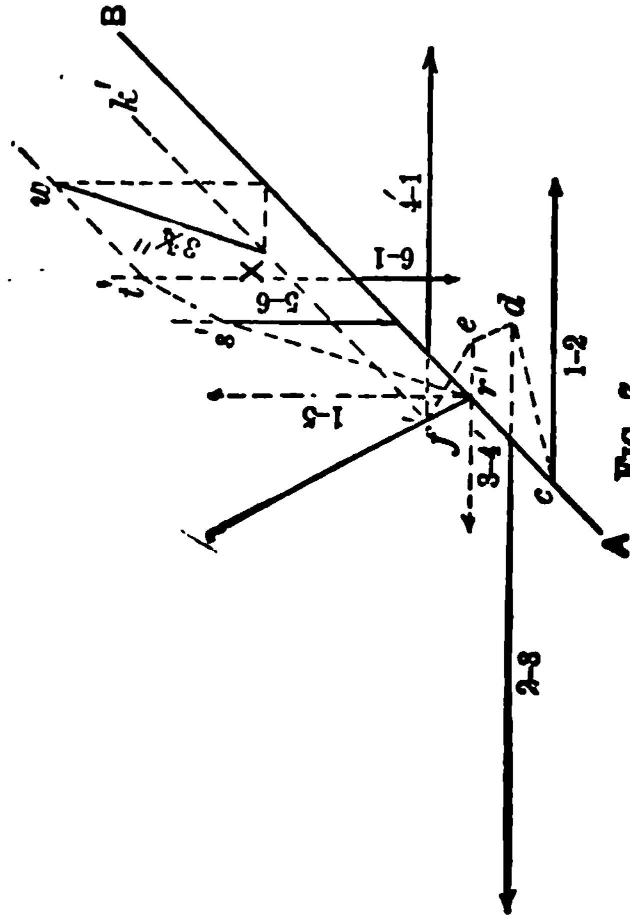


FIG. 7.

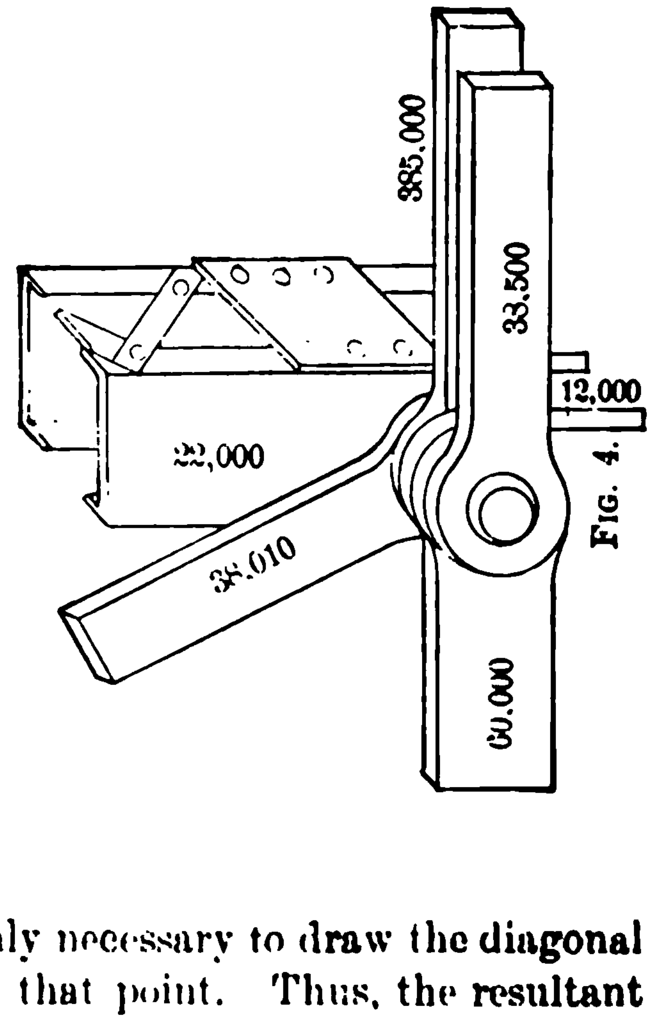


FIG. 4.

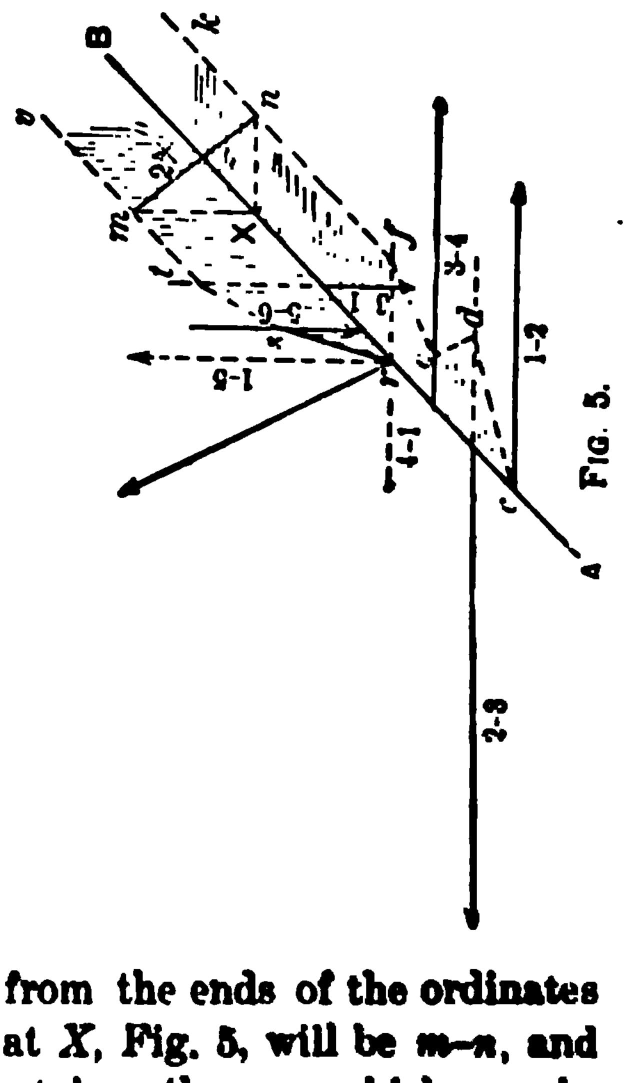


FIG. 5.

only necessary to draw the diagonal from the ends of the ordinates at that point. Thus, the resultant at X, Fig. 5, will be m–n, and it is evident that this is the longest hypothenuse which can be

drawn; and this hypothenuse, multiplied by 0–1 (20,000 pounds), gives 52,500 pounds as the maximum bending moment on the pin.

To obtain the maximum bending moment, it is necessary to take the longest hypothenuse that can be drawn, no matter at what place it occurs.

If one desires to try the effect of changing the order of the bars on the pin, it can readily be done. Suppose the diagonal tie to change places with the next chord bar. The horizontal stress diagram then becomes 1–2, 2–3, 3–4′, 4′–1. The equilibrium polygons

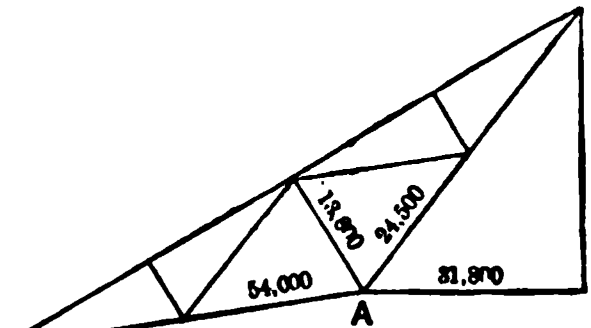


FIG. 8.

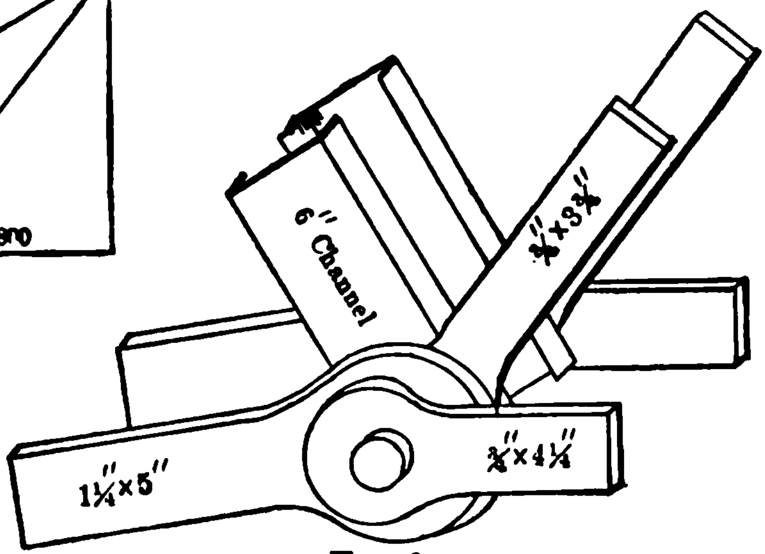


FIG. 9.

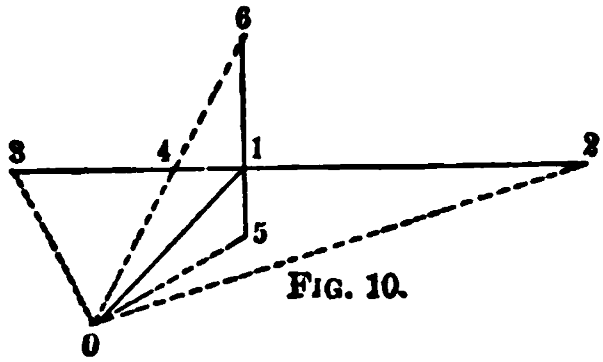


FIG. 10.

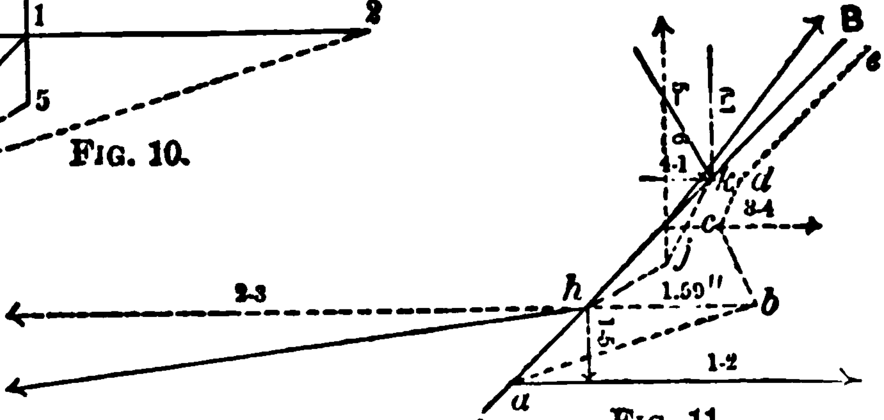


FIG. 11.

will now be (Fig. 7) $c\ d\ e\ f'\ k'$ and $r'\ s'\ t'\ w$, and the longest hypothenuse, $w\ x$, or 3¾", which makes the bending moment 75,000 pounds, showing that the arrangement in Fig. 4 is the best.

As a rule, in arranging the bars on a pin, those forces which counteract each other should be close together.

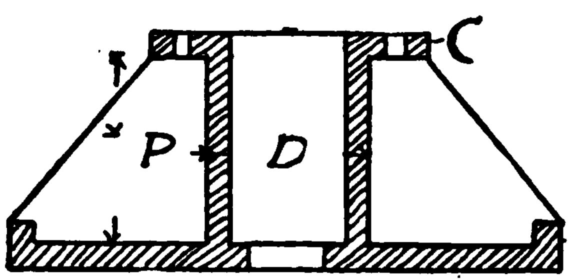
To further illustrate this method of determining the bending moment on pins, we will determine the bending moment for the pin at the joint A, Fig. 8. This is the same truss as worked out on page 585, the strains given in Fig. 8 being ½ of the strains at the joint, as all the pieces are doubled. Fig. 9 shows the size and arrangement of the ties and strut. It is assumed that the web of

the channel is reënforced to make it $\frac{5}{8}''$ thick. Drawing the line *AB*, Fig. 11, we lay off the outer force at *a*; then measuring off an inch, the distance between centres of the two outer bars, we lay off the next force parallel to the direction in which it acts; and in the same way, the other two forces. The three inclined forces must be resolved into their horizontal and vertical components. We next draw the stress diagram (Fig. 10) to the same scale of pounds, making 1 0 equal 20,000 pounds. The lines 0 4 and 0 6 happen, in this case, to coincide. Then, in Fig. 11, we draw *a b* parallel to 0 2, *b c* parallel to 0 3, *c d* = 0 4, and *d e* parallel to 0 1. In the same way, we obtain the line *h j k B*. In this case, it will be seen that the longest horizontal ordinate is *h b*, while at that point there is no vertical ordinate; also, that no hypothenuse can be drawn which will be as long as *h b*, so that we must take *h b* as the greatest resultant; and this, multiplied by 20,000 pounds, gives 31,800 pounds as the maximum bending moment on the pin. It will be seen that this is just the product of the outer force by its arm to the centre of the next bar, so that the greatest bending moment is at that point.

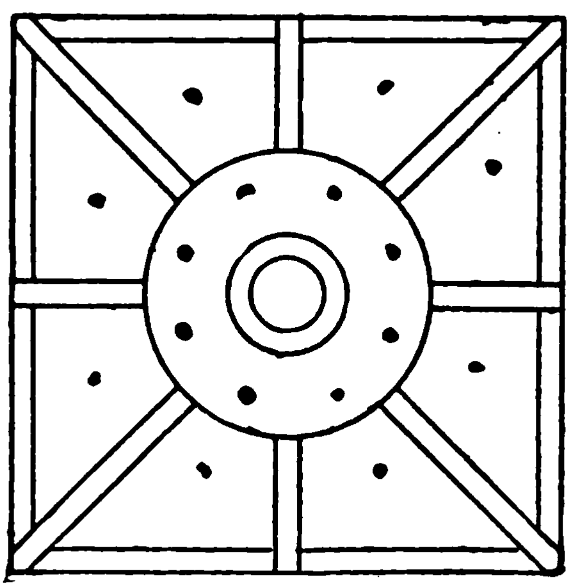
To determine the size of the pin, we find, from Table III., that for a steel pin to sustain this moment, allowing a fibre strain of 20,000 pounds, we shall need a $2\frac{5}{8}''$ pin. This pin has a bearing value of 31,500 pounds for a bar an inch thick. The outer bar in this case is $\frac{3}{4}''$ thick, and has a strain of 31,800 pounds, equivalent to 42,400 pounds for a 1″ bar. And we see, from Table II., that we shall need to use a $3\frac{1}{2}''$ pin to meet this strain. The shearing strength of a $3\frac{1}{2}''$ pin is 36 tons, or more than double the strain. Hence we must use a $3\frac{1}{2}''$ pin, or, by increasing the thickness of the bars, we might reduce the pin to 3 inches.

PROPORTIONS OF CAST-IRON BEARING-PLAT FOR GIRDERS AND COLUMNS (1895).

If a heavily loaded column or girder should rest directly up wall or pier of masonry, the weight would be distributed over a small area that in most cases there would be danger of crus the masonry, particularly if it were of brick or rubble work.



Section



Plan

FIG 1

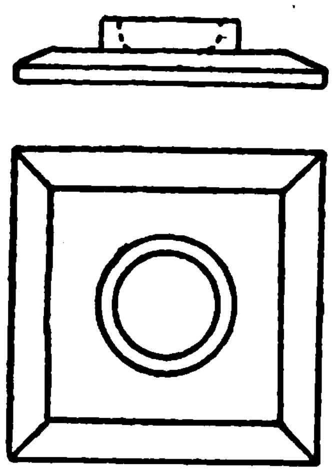


FIG 2

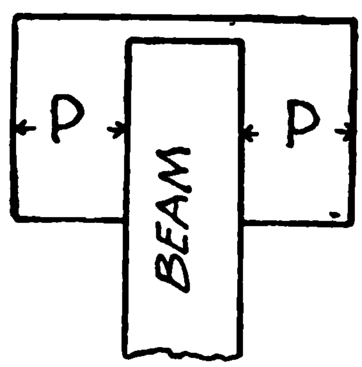


FIG 3

prevent this, it is customary to put a bearing-plate between end of the beam or column and the masonry, the size of the] being such that the load from the column or girder divided by area of the plate shall not exceed the safe crushing-strength o masonry per unit of measurement.

The load per square inch on different kinds of masonry not exceed the following limits :

For granite	1,000	lbs. per sq. in.
" best grades of sandstone	700	" " " "
" soft sandstone	400	" " " "
" extra hard brickwork in cement mortar	150 to 170	" " " "
" good hard brickwork in lime mortar	120	" " " "
" good Portland cement concrete	150	" " " "
" sand or gravel	60	" " " "

Example 1.—The basement columns of a six-story warehouse support a possible load of 212,000 pounds each ; under the column is a base-plate of cast-iron, resting on a bed of Portland cement concrete two feet thick: What should be the dimensions of the base-plate ?

Answer.—As the plate rests on concrete, the bottom of the plate should have an area equal to $212,000 \div 150 = 1,413$ square inches, or 37 inches square. The column should be about 10 inches in diameter and 1 inch thick. The shape of the base-plate should be as shown in Fig. 1.

The height K should be equal to the projection P, and D should be equal to the diameter of the column. The thickness of all portions of the plate should be equal to that of the column above the base. This is not so much required for strength as to get a perfect casting, as such castings are liable to crack by unequal cooling when the parts are of different thicknesses. The projection of the flange C should be three inches, to permit of bolting the plate to the bottom of the column. It will be seen that in such a plate no transverse strain is developed in any portion of it.

THICKNESS OF FLAT BASE-PLATES.

For small columns and wooden posts with light loads, plain flat iron plates are generally used. They may have a raised ring to fit inside the base of an iron column, or for a wooden post, a raised dowel, 1½ inches or 2 inches in diameter. If the plate is very thick, a saving in the weight of the plate may be made by bevelling the edge, as shown in Fig. 2, without loss of strength. The outer edge, however, should not be less than one inch thick.

When such a plate is used, it is evident that if the plate is to distribute the load equally over its entire area, it must have sufficient transverse strength to resist bending or breaking, and this strength will depend upon the thickness of the plate. It is difficult to make an exact formula for the thickness of such plates,

but the writer has derived the following formula, which he believes will be always on the safe side and without any great excess of strength:

Thickness of plate in inches [illegible]

in which w = [illegible] inches, and P [illegible] post or column. [illegible] we have w = [illegible] pounds and P = [illegible]

[illegible]

Therefore [illegible] $4\frac{1}{2}$ inches [illegible]

Example [illegible] plate [illegible] What should [illegible]

Answer.—[illegible]

Thickness = [illegible]

The [illegible] $1\frac{1}{2}$ inches.

[illegible]

Base-plates [illegible] half [illegible] down [illegible] bottom [illegible] evenly [illegible]

[illegible]

The [illegible] plates [illegible] the [illegible]

The [illegible] break [illegible] same [illegible]

Thickness [illegible]

multiplied by 7,000, gives 42,000 pounds as the safe strength of one bracket.

The resistance to crushing may be found by multiplying the distance X by the thickness of the bracket and the product by 13,000. Thus, if X is four inches and the thickness one inch, the resistance to crushing would be 52,000 pounds. Such a bracket would support the end of a 20-inch light steel beam of 16 feet span under its full load; for heavier beams, the thickness of the bracket and also the length D should be increased.

CHAPTER XI.

STRENGTH OF POSTS, STRUTS, AND COLUMNS.

As the strength of a post, strut, or column, depends primarily upon the resistance of the given material to crushing, we must first determine the ultimate crushing-strength of all materials used for this purpose.

The following table gives the strength for all materials used in building, excepting brick, stone, and masonry, which will be found in Chap. VI.

TABLE I.

Average Ultimate Crushing-Loads, in Pounds per Square Inch, for Building-Materials.

MATERIAL.	Crushing weight, in lbs. per sq. inch.	MATERIAL.	Crushing weight, in lbs. per sq. inch.
	C.		*C.*
For STONE, BRICK, and MASONRY, see Chap. VI.		WOODS (*continued*).	
		Beech	9,300 a
		Birch	11,600 a
		Cedar	6,500 a
METALS.		Hemlock	5,400 b
Cast-iron	80,000	Locust	11,720 b
Wrought-iron . . .	36,000	Black walnut . . .	5,690
Steel (cast)	225,000 a	White oak	3,150 to 7,000
		Yellow pine . . .	4,400 to 6,000
WOODS.		White pine	2,800 to 4,500
Ash	8,600 a	Spruce	2,800 to 4,500

The values given for wrought and cast iron are those generally used, although a great deal of iron is stronger than this. The values for white oak, yellow pine, and spruce, are derived from experiments on full-size posts, made with the government testing-machine at Watertown, Mass.; the smaller value representing the strength of such timber as is usually found in the market, and the larger value, the strength of thoroughly seasoned straight-grained timber. For these woods a smaller factor of safety may be

a Trautwine. b Hatfield.

used than for the others, the strength of which was derived from experiments on small pieces.

The values for wood are for dry timber. Wet timber is only about one-half as strong to resist compression as dry timber, and this fact should be taken into account when using green timber.

The strength of a column, post, or strut depends, in a large measure, upon the proportion of the length to the diameter or least thickness. Up to a certain length, they break simply by compression, and above that they break by first bending sideways, and then breaking.

Wooden Columns.

For wooden columns, where the length is not more than twelve times the least thickness, the strength of the column or strut may be computed by the rule,

$$\text{Safe load} = \frac{\text{area of cross-section} \times C}{\text{factor of safety}}. \quad (1)$$

where C denotes the strength of the given material as given in Table I.

The factor of safety to be used depends upon the place where the column or strut is used, the load which comes upon it, the quality of the material, and, in a large measure, upon the value taken for C.

Thus for white oak, yellow pine, and spruce, the value C is the actual crushing-strength of full-size posts of ordinary quality: hence we need not allow a factor of safety for these greater than four. For the other woods, we should use a factor of safety of at least six.

If the load upon the column or post is such as comes upon the floor of a machine-shop, or where heavy machinery is used, or if the strut is for a railway-bridge, a larger factor of safety should be used in all cases.

If the quality of the timber is exceptionally good, we may use the larger values for the constant C, in the case of the last four woods given in the table. For ordinary hard pine or oak posts, multiply the area of cross-section in inches by 1000; for spruce, by 800, and for white pine, by 700 pounds.

EXAMPLE I. — What is the safe load for a hard-pine post 10 by 10 inches, 12 feet long?

Ans. Area of cross-section = 10 × 10 = 100 square inches; 100 × 1000 = 100,000 pounds.

EXAMPLE II.—What is the safe load for a spruce strut 8 feet long. 6″ × 8″ ?

Ans. Area of cross-section = 48 ; 48 × 800 = 38,400 pounds.

Strength of Wooden Posts over Twelve Diameters in Length.

When the length of a post exceeds twelve times its least thickness or diameter, the post is liable to bend under the load, and hence to break under a less load than would a shorter column of the same cross-section.

To deduce a formula which would make the proper allowance for the length of a column has been the aim of many engineers, but their formulæ have not been verified by actual results.

Until within two or three years the formulæ of Mr. Lewis Gordon and Mr. C. Shaler Smith have been generally used by engineers, but the extensive series of tests made on the Government testing machine at Watertown, Mass., on full-size columns, show that these formulæ do not agree with the results there obtained.

Mr. James H. Stanwood, Instructor in Civil Engineering, Mass. Institute Technology, in the year 1891 platted the values of all the tests made at the Watertown Arsenal up to that time on full-size posts. From the drawing thus obtained he deduced the following formula for *yellow pine* posts:

$$\text{Safe load per square inch} = 1{,}000 - 10 \times \frac{\text{length in ins.}}{\text{breadth in ins.}}$$

The author has carefully compared this formula with the results of actual tests, and with other formulæ, and believes that it meets the actual conditions more nearly than any other formula, and he has therefore discarded the tables of wooden posts given in the previous editions of this work and prepared the following tables for the strength of round and square posts of sizes coming within the range of actual practice.

For other sizes the loads can easily be computed by the formula.

The loads for oak and white pine posts were computed by the following formulæ :

For oak and Norway pine:

$$\text{Safe load per square inch} = 750 - 7.5 \times \frac{\text{length in ins.}}{\text{breadth in ins.}}$$

For white pine and spruce posts:

$$\text{Safe load per square inch} = 625 - 6 \times \frac{\text{length in ins.}}{\text{breadth in ins.}},$$

in which the *breadth* is the least side of a rectangular strut, or the diameter of a round post. The round posts were computed for the half-inch, to allow for being turned out of a square post, of the size next larger.

The formulæ were only used for posts exceeding 12 diameters for yellow pine, and ten diameters for other woods.

For posts having bad knots, or other defects, or which are known to be eccentrically loaded, a deduction of from 10 to 25 per cent. should be made from the values given in the tables.

SAFE LOAD IN POUNDS FOR YELLOW PINE POSTS (ROUND AND SQUARE).

SAFE LOAD IN POUNDS FOR OAK AND NORWAY PINE POSTS (ROUND AND SQUARE).

8
4
6
6
6
6
7
8
8
8
9
1
1
1
1
1
5

SAFE LOAD IN POUNDS FOR WHITE PINE AND SPRUCE POSTS (ROUND AND SQUARE).

Size of in Inc
4 × 6. .
5½ Roun
6 × 6 ..
6 × 8. .
6 × 10 ..
7½ Roun
8 × 8
8 × 10 ..
8 × 12 ..
9½ Roun
10 × 10.
10 × 12.
10 × 14 .
11½ Rou
12 × 12..
12 × 14.
12 × 16
14 × 14..
16 × 16..
18 × 18.
20 × 20 .

Eccentric Loading.

When the load on a post is applied in such a way that it is not distributed uniformly over the end of the post, the loading is called eccentric and the effect on the post is much more injurious than if the load were uniformly distributed. When a post supports a girder on one side only, or when the weight from one girder is much more than from the other, the load becomes eccentric, and an allowance must be made in the safe load varying from 10 to 25 per cent., according to the amount of eccentricity.

The exact allowance cannot be calculated, so that one must necessarily use his judgment in the matter, remembering that it is best to be on the safe side.

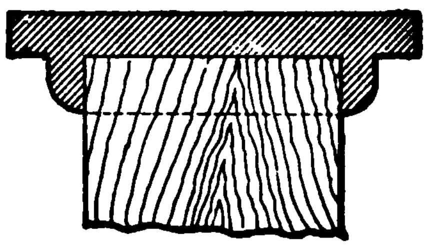


Fig. 1.

Iron caps for timber pillars are often used in important constructions, and are an excellent invention, as they serve to distribute the thrust evenly through the pillar, and also form a bracket, which is often desirable, for supporting the ends of girders where a second post rests on top of the first. Fig. 1 shows the section of one of the simplest forms of caps.

The Goetz and Duvinage caps, described at the end of Chapter XXIV., are the best shape for mill construction.

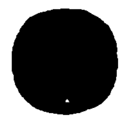
Cast-Iron Columns.

For cast-iron columns, where the length is not more than six or eight times the diameter or breadth of column, the safe load may be obtained by simply multiplying the metal area of cross-section by $6\frac{3}{4}$ tons, which will give tons for the answer.

Above this proportion, that is, where the length is *more* than eight times the breadth or diameter, the following formulas should be used. These formulas are known as Gordon's and Rankine's.

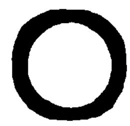
FORMULAS—

For solid cylindrical cast-iron columns,



$$\text{Safe load in lbs.} = \frac{\text{Metal area} \times 13330}{1 + \dfrac{\text{sq. of length in inches}}{\text{sq. of diam. in inches} \times 266}}. \quad (4)$$

For hollow cylindrical columns of cast-iron,

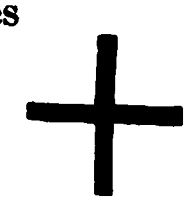


$$\text{Safe load in lbs.} = \frac{\text{Metal area} \times 13330}{1 + \dfrac{\text{sq. of length in inches}}{400 \times \text{sq. of diam. in inches}}}. \quad (5)$$

For hollow or solid rectangular pillars of cast-iron,

$$\text{Safe load in lbs.} = \frac{\text{Metal area} \times 13330}{1 + \dfrac{\text{sq. of length in inches}}{500 \times \text{sq. of least side in inches}}}. \quad (6)$$

For cast-iron posts, the cross-section being a cross of equal arms.



$$\text{Safe load in lbs.} = \frac{\text{Metal area} \times 13330}{1 + \dfrac{\text{sq. of length in inches}}{133 \times \text{sq. of total breadth in inches}}}. \quad (7)$$

EXAMPLE I.—What is the safe load for a hollow cylindrical cast-iron column, 10 feet long, 6 inches external diameter, and 1″ thickness of shell?

Ans. We must first find the metal area of the cross-section of the column, which we obtain by subtracting the area of a circle of four inches in diameter from the area of one six inches in diameter. The remainder will be the area of the metal. The area of a six-inch circle is 28.27 square inches, and of a four-inch, 12.56 square inches; and the metal area of the column is 15.71 square inches.

Then, substituting known values in formula 5, we have

$$\text{Safe load} = \frac{15.71 \times 13330}{1 + \dfrac{14400}{400 \times 36}} = 104700 \text{ pounds.}$$

There is no use in carrying the result farther than the nearest hundred pounds, because the accuracy of our formulas will not warrant it.

EXAMPLE II. — What is the safe load for a cast-iron column 12 feet long, the cross-section being a cross with equal arms, one inch thick, the total breadth of two arms being 8″?

Ans. The area of cross-section would be $8 + 7 = 15$ square inches. Then, by formula 7,

$$\text{Safe load in lbs.} = \frac{15 \times 13330}{1 + \dfrac{20736}{133 \times 64}} = 58300 \text{ pounds.}$$

Projecting Caps.

Hollow columns calculated by the foregoing formulas should not be cast with heavy projecting mouldings round the top or bottom,

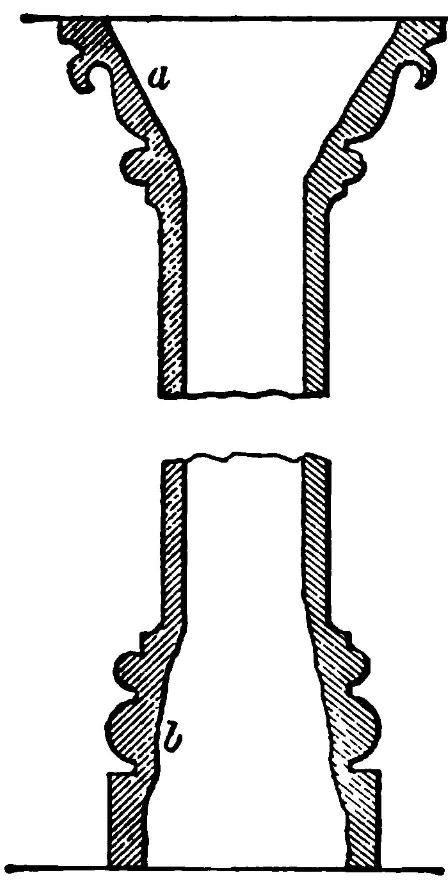


Fig. 2

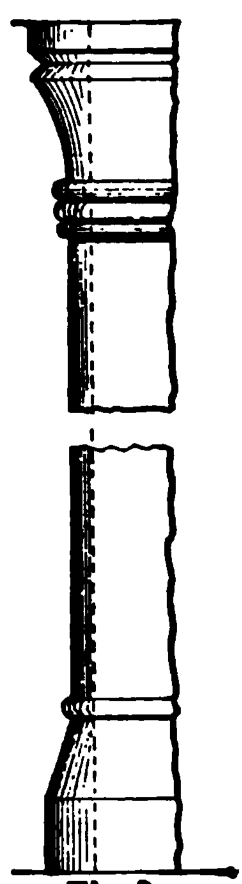


Fig. 3

as in Fig. 2, at *a* and *b*. It is obvious that these are weak, and would break off under a load much less than would be required to

crush the column. When such projecting ornaments are deemed necessary, they should be cast separately, and be attached to a prolongation of the column by iron pins or screws. Ordinarily it is better to adopt a more simple base and cap, which can be cast in one piece with the pillar, without weakening it, as in Fig. 3.

In all the rules and formulas given for cast-iron columns, it is supposed that the ends have bearings planed true, and at right angles to the axis of the column.

When the columns are used in tiers, one above the other, the end connections of the columns should be made by projecting flanges, wide enough to receive ¾-inch bolts for bolting the columns together, as shown in Fig. 4, page 242*e*, and the entire ends and flanges should be turned true to the axis of the column. The end joints are generally placed just above the floor beams, for convenience in erecting the work.

The basement columns should be bolted to cast-iron base plates, as shown in Fig. 1, page 242*a*. The author does not consider it advisable to use cast-iron columns with hinged ends, or in buildings whose height exceeds twice their width.

Tables of Cast-Iron Columns.

By an inspection of the foregoing formulas for cast-iron columns, it will be seen, that, all other conditions being the same, the strength per square inch of cross-section of any column varies only with the ratio of the length to the diameter or least thickness. Thus a column 15 feet long and 10 inches diameter would carry the same load per square inch as a similar column 9 feet long and 6 inches diameter, both having the ratio of length to diameter as 18 to 1.

Owing to this fact, tables can be prepared giving the safe load per square inch for columns having their ratio of length to diameter less than 40.

On this principle Table IV. has been computed, giving the loads per square inch of cross-section for hollow cylindrical and rectangular cast-iron columns.

To use this table, it is only necessary to divide the length of the column in inches by the least thickness or diameter, and opposite the number in column I. coming nearest to the quotient find the safe strength per square inch for the column. Multiply this load by the metal area in the cross-section of the column, and the result will be the safe load for the column.

Example III. — What is the safe load for a 10-inch cylindrical cast-iron column 15 feet long, the shell being 1 inch thick?

Ans. The length of the column divided by the diameter, both in inches, is 18, and opposite 18 in Table IV. we find the safe load

per square inch for a cylindrical column to be 7,366 pounds. The metal area of the column we find to be 28.27 inches; and, multiplying these two numbers together, we have for the safe load of the column 208,236 pounds, or about 104 tons.

Besides this table, we have computed Table V. following, which gives at a glance the safe load for a cast-iron column coming within the limits of the table, and of a thickness there shown.

Thus, to find the safe load for the column given in the last example, we have only to look in the table for columns having a diameter of 10 inches and a thickness of shell of 1 inch, and opposite the length of the column we find the safe load to be 104 tons, the same as found above.

The safe load in both tables is *one-sixth* of the breaking-load.

TABLE IV.

Strength of Hollow Cylindrical or Rectangular Cast-Iron Pillars.

(Calculated by Formulas 5 and 6.)

Length divided by external breadth or diameter.	Breaking-weight in pounds per square inch.		Safe load in pounds per square inch.	
	Cylindrical.	Rectangular.	Cylindrical.	Rectangular.
5	75,294	76,190	12,549	12,698
6	73,395	74,627	12,232	12,438
7	71,269	72,859	11,878	12,143
8	68,965	70,922	11,494	11,820
9	66,528	68,846	11,088	11,474
10	64,000	66,666	10,666	11,111
11	61,420	64,412	10,236	10,735
12	58,823	62,111	9,804	10,352
13	56,239	59,790	9,373	9,965
14	53,859	57,471	8,976	9,578
15	51,200	55,172	8,533	9,195
16	48,780	52,910	8,130	8,817
17	46,444	50,697	7,741	8,449
18	44,198	48,543	7,366	8,090
19	42,050	46,457	7,008	7,743
20	40,000	44,444	6,666	7,407
21	38,050	42,508	6,341	7,085
22	36,200	40,650	6,033	6,775
23	34,455	38,872	5,742	6,479
24	32,787	37,174	5,464	6,195
25	31,219	35,555	5,203	5,926
26	29,741	34,014	4,957	5,669
27	28,343	32,547	4,724	5,423
28	27,027	31,152	4,504	5,192
29	25,785	29,828	4,297	4,971
30	24,615	25,571	4,102	4,761
31	23,512	27,310	3,918	4,818
32	22,472	26,246	3,745	4,374
33	21,491	25,172	3,581	4,195
34	20,565	24,154	3,427	4,026
35	19,692	23,188	3,282	3,864

TABLE V.

Showing Safe Load in Tons for Cylindrical Cast-Iron Columns.

Thickness of Shell ¾ Inch.

Length of column.	Diameter of column (outside).							
	6 ins.	7 ins.	8 ins.	9 ins.	10 ins.	11 ins.	12 ins.	13 ins.
Feet.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.
6	60.6	78.1	94.0	110.8	128.6	144.9	161.7	180.0
7	55.7	72.2	88.9	106.9	124.2	140.1	156.4	176.0
8	50.7	66.3	83.8	101.1	117.7	135.2	151.1	170.3
9	45.8	61.9	78.7	95.2	113.4	130.4	145.8	164.5
10	40.8	56.0	73.5	89.4	106.8	123.2	140.5	158.7
11	37.1	51.5	68.4	83.6	100.1	118.3	135.2	153.0
12	33.4	47.1	63.3	79.7	95.9	113.5	129.9	147.2
13	30.9	44.2	58.1	73.9	89.4	106.3	124.6	141.4
14	27.2	39.8	54.7	70.0	85.0	101.4	119.2	135.6
15	24.7	36.8	49.6	64.1	78.5	96.6	114.0	129.9
16	22.3	33.9	46.2	60.3	71.9	91.8	108.7	124.1
18	-	29.0	41.0	52.5	67.6	84.5	103.4	118.3
20	-	24.4	36.0	44.7	63.3	77.2	98.1	112.5
	Metal area of cross-section.							
	sq. ins.	sq. ins.	sq. ins.	sq. ins.	sq. ins.	sq. ins.	sq. ins.	sq. ins.
	12.37	14.73	17.10	19.44	21.80	24.15	26.51	28.86

Thickness of Shell 1 Inch.

Length of column.	Diameter of column (outside).							
	6 ins.	7 ins.	8 ins.	9 ins.	10 ins.	11 ins.	12 ins.	13 ins.
Feet.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.	Tons.
6	77	100	121	143	167	188	211	234
7	71	92	118	138	161	182	204	230
8	64	85	108	131	153	176	197	222
9	58	79	101	123	147	170	190	215
10	52	72	95	116	138	161	183	207
11	47	66	88	108	130	154	175	200
12	42	60	81	102	124	147	169	192
13	39	57	75	95	116	138	162	184
14	35	52	69	90	110	132	155	177
15	31	47	64	83	104	126	148	170
16	28	43	59	78	96	119	142	162
18	25	39	53	68	88	105	128	151
20	22	35	46	58	79	94	114	136
	Metal area of cross-section.							
	sq. ins.	sq. ins.	sq. ins.	sq. ins.	sq. ins.	sq. ins.	sq. ins.	sq. ins.
	15.71	18.82	22.00	25.14	28.27	31.41	34.56	37.70

The principal disadvantage, as found in practice, is the difficulty, if not impossibility, of making rigid connections with the beams and girders. In buildings of not more than five or six stories, however, this is not of great importance.

Cast-iron is, of course, subject to flaws, and the columns are liable to be cast of uneven thickness of metal, but by careful inspection these defects can be discovered, and any columns containing them rejected.

For unprotected columns, cast-iron is unquestionably better than steel, as has been quite conclusively demonstrated by the experiments of Prof. Bauschinger, of Munich. Cast-iron, three quarters of an inch or more in thickness, is also practically uninjured by rust, while it is claimed that wrought-iron or steel may be almost destroyed by it.

Although cast-iron columns may be made in a great variety of shapes, the hollow cylindrical and rectangular columns have thus far been the principal shapes used, and for interior unprotected columns the cylindrical column probably meets the usual requirements better than any others. Every year, however, the requirements of building regulations are being made more strict, so that at the present time it is required in most of our large cities that all vertical supports in buildings over five stories in height shall be protected by fireproof material, and for such buildings the author would call attention to the H-shaped column, as offering the following advantages :

1. Being entirely open, with both the interior and exterior surfaces exposed, any inequalities in thickness can be readily discovered, and the thickness itself easily measured, thus obviating any necessity for boring, and rendering the inspection of the columns much less tedious.

2. The entire surface of the column can be protected by paint.

3. When built in brick walls, the masonry fills all voids, so that no open space is left, and if the column is placed as shown in Fig. 4, only the edge of the column comes near the face of the wall.

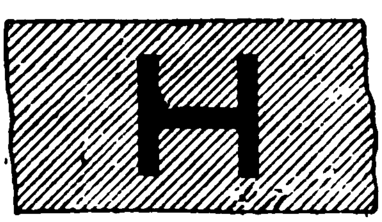


FIG. 4.

4. Lugs and brackets can be cast on such columns better than on circular columns, especially for wide and heavy girders.

5. The end connections of the columns do not require projecting rings, or flanges, which are often objectionable in circular columns.

The cost of columns of this shape should not exceed that of circular columns of the same strength.

As to the strength of such columns, the only experimental data which we have on the subject is that obtained from the experiments of Mr. Eaton Hodgkinson, which give them about the same strength as cylindrical columns of the same diameter, when the length does not exceed thirty diameters and the thickness is not less than three-quarters of an inch. When surrounded by masonry they would probably be stronger than the cylindrical column.

The column may be fireproofed in the same way as the Z-bar column, which it much resembles. The space occupied by the column slightly exceeds that of both the cylindrical and Z-bar column,

FIG. 5. FIG. 6.

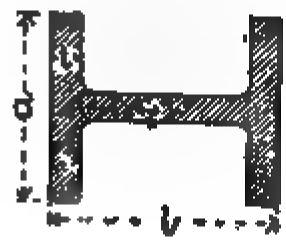
but not enough to be of any serious consequence. Figs. 5 and 6 show details of end connections and brackets, and of base plate.

The beams running at right angles to the web should be tied together by wrought-iron straps passing through holes in the web of the column.

The following table has been calculated with the same stress per square inch of metal as allowed for the columns in Table V.

TABLE V.a

Safe Loads in Tons of 2,000 Pounds for H-shaped



								16	[illegible]	20
" " 1½	19½	64	52	48	49					
6 × 8 × ¾	13½	45	37	34	30					
" " 1	16	59	48	45	39					
" " 1¼	21½	72	59	54	49					
7 × 7 × 1	19	72	60	57	54	47				
" " 1¼	23½	88	71	69	66	58				
7 × 9 × 1	21	80	65	63	60	53				
" " 1¼	25½	97	79	77	72	64				
8 × 8 × ¾	16½	73	60	55	52	48	45			
" " 1	22	98	79	72	68	64	59			
" " 1¼	26½	114	96	88	83	76	72			
8 × 10 × 1	24	102	86	79	74	69	65			
" " 1¼	29½	125	105	97	91	85	79			
" " 1½	34½	146	124	114	107	100	93			
9 × 9 × 1	25	116	100	96	90	83	75	67		
" " 1¼	30½	142	122	118	110	109	93	83		
" " 1½	36	167	144	138	[illegible]	120	108	97		
9 × 10 × 1	26	121	104	100	94	86	78	70		
" " 1¼	31½	148	127	123	115	106	96	86		
" " 1½	37½	174	150	144	135	125	112	101		
10 × 10 × 1	29	137	123	116	109	103	96	86		
" " 1¼	34½	168	151	142	134	126	118	104		
" " 1½	40½	198	178	168	158	149	139	123		
" " 1¾	46½	227	204	192	180	170	160	141		
10 × 12 × 1	30	147	132	124	117	110	103	91		
" " 1¼	36½	180	162	153	144	135	127	112		
" " 1½	43½	213	191	180	169	160	150	132		
" " 1¾	49½	244	219	207	194	183	173	151		
" " 2	56	274	246	232	218	206	193	170		
12 × 12 × 1	34	180	166	150	152	144	136	125	118	
" " 1¼	41½	221	205	196	187	178	170	154	139	
" " 1½	49½	262	242	232	221	210	201	180	164	
" " 1¾	56½	301	278	266	254	242	231	209	189	
" " 2	64	339	313	300	286	273	260	236	213	
12 × 14 × 1¼	44½	235	217	208	198	189	180	[illegible]	147	
" " 1½	52½	278	257	246	235	223	213	193	174	
" " 1¾	60½	320	295	283	270	257	245	222	201	
" " 2	68	360	332	318	304	288	276	250	226	
" " 2¼	75½	399	369	353	337	321	306	277	250	

Hollow Rectangular Cast-iron Columns.

The increasing use of hollow rectangular cast-iron columns in buildings, particularly when enclosed in brick walls, has led the author to compute Table V.*b*, which gives the safe loads for a large number of sizes and lengths, the application of the table being readily apparent. The loads correspond with and are based upon those given in the last column of Table IV.

The author would recommend that the various sizes be not used for greater lengths than those given in the table.

TABLE V.b

Safe Loads in Tons of 2,000 Pounds for Hollow Rectangular Cast-iron Columns.

Outside Dimensions. Ins.	Thickness of Shell. Ins.	Sectional Area in Ins.	Length of Column in Feet. 10	12	13	14	15	16	18	20
6 × 6	¾	15¾	58	48	44	40				
" "	1	20	74	61	56	51				
" "	1¼	23¾	87	73	66	61				
6 × 8	¾	18¾	69	58	52	48				
" "	1	24	88	74	67	62				
" "	1¼	28¾	106	88	80	74				
6 × 10	¾	21¾	80	67	61	56	51			
" "	1	28	103	86	78	72	66			
" "	1¼	33¾	124	104	94	87	80			
7 × 7	¾	18¾	78	67	62	58	53			
" "	1	24	100	86	80	74	68			
7 × 9	¾	21¾	91	78	72	67	62			
" "	1	28	117	100	93	86	79			
8 × 8	¾	21¾	100	87	81	76	71	65		
" "	1	28	128	112	105	98	92	84		
" "	1¼	33¾	155	135	126	118	110	101		
8 × 10	¾	24¾	113	99	92	86	80	74		
" "	1	32	147	128	120	112	105	96		
" "	1¼	38¾	178	155	145	135	125	116		
8 × 12	¾	27¾	127	111	104	97	90	83		
" "	1	36	165	144	135	126	117	108		
" "	1¼	43¾	201	175	164	153	144	125		
10 × 10	¾	27¾	143	130	123	117	111	105	94	
" "	1	36	186	169	160	151	144	136	122	
" "	1¼	43¾	226	205	194	184	175	166	148	
" "	1½	51	263	239	227	215	204	193	172	
10 × 12	¾	30¾	159	144	137	129	122	116	104	
" "	1	40	206	188	178	168	160	152	136	
" "	1¼	48¾	252	229	217	205	195	185	165	
" "	1½	57	293	267	253	240	228	216	193	
10 × 14	¾	33¾	174	158	150	142	135	128	114	
" "	1	44	227	206	196	185	176	167	149	
10 × 16	1	48	248	225	214	202	192	182	163	
10 × 18	1	52	268	244	231	219	208	197	176	
10 × 24	1	64	330	300	285	270	256	243	217	
12 × 12	¾	33¾	187	174	168	161	154	148	136	124
" "	1	44	244	227	219	210	201	193	177	162
" "	1¼	53¾	298	278	267	256	246	236	217	198
" "	1½	63	349	325	312	300	289	277	254	233
12 × 14	¾	36	203	189	182	175	168	161	148	136
" "	1	48	266	248	239	229	220	211	193	177
12 × 16	1	52	288	268	258	248	238	228	210	192
12 × 24	1	68	374	351	338	325	312	299	274	251
14 × 16	1	56	330	308	297	288	278	268	250	235
16 × 16	1	60	354	336	330	324	318	310	291	275
16 × 18	1	64	377	358	352	345	339	330	314	298
18 × 18	1	68	414	401	391	380	374	367	346	328
18 × 24	1	80	488	472	460	448	440	428	406	388

Wrought-Iron and Steel Columns and Struts (1891).

Within the past three years wrought-iron and steel columns have been gradually taking the place of cast-iron columns in fire-proof buildings, and the time is probably not far distant when wrought-iron or steel columns will be used almost exclusively for the interior supports of all large buildings.

In iron or steel trusses the struts are invariably made of the same material, though, of course, the strut bars are of a different section from that used for ties.

There are many contingencies which may arise in the manufacture of cast-iron columns which preclude anything approaching uniformity in the product.

Among these are unevenness in the thickness of the metal, which has sometimes been found to be very different on one side of a round column from that on the opposite side. The presence of confined air, producing "blow holes" and "honey-comb," and the col-

lection of impurities at the bottom of the mould are also frequent sources of weakness in cast iron.

The most critical condition, however, is that due to the unequal contraction of the metal during the process of cooling, thereby giving rise to initial strains, at times of sufficient force to produce rupture in the column or in its lugs on the slightest provocation.

In many cases the trouble is due to faulty designing or carelessness in the execution of the work; yet, even under favorable conditions, it is so difficult to secure equal radiation from the moulds in all directions that castings entirely exempt from inherent shrinkage strains are probably seldom produced.

As a protection against these contingencies, resort must be had either to the uncertain expedient of a high factor of safety, or a material such as wrought iron or rolled steel must be adopted of a more uniform and reliable character than cast iron.

Columns built up of rolled sections also offer better facilities for fire-proof covering ; and for columns where extreme loads are to be supported, as in the lower stories of very high buildings, wrought-iron and steel columns will occupy less room than a cast-iron column, and in many instances will be found to be cheaper.

The forms of rolled columns now in general use in buildings are the "Phœnix," "Larimer," "Gray," and "Z-bar" columns, illustrated on pages 267–289*h*.

For the strut bars of trusses two-channels bars, angle or T-bars, are generally used.

In trusses with pin connections the channel bar offers the best shape for the struts. I-beams are also often used.

Strength of Wrought-iron Posts.

The formulæ most generally accepted by engineers of the present day for the strength of irregular-shaped sections (such as nearly all these struts are) are as follows :

Column—Square Bearing,

$$\left.\begin{array}{l}\text{Ultimate strength}\\ \text{in lbs. per sq. inch}\end{array}\right\} = \frac{40{,}000}{1 + \dfrac{\text{sq. of length in inches}}{36{,}000 \times r^2}} \qquad (8)$$

Column—Pin and Square Bearing,

$$\left.\begin{array}{l}\text{Ultimate strength}\\ \text{in lbs. per sq. inch}\end{array}\right\} = \frac{40{,}000}{1 + \dfrac{\text{sq. of length in inches}}{24{,}000 \times r^2}} \qquad (9)$$

Column—Pin Bearing,

$$\left.\begin{array}{l}\text{Ultimate strength}\\ \text{in lbs. per sq. inch}\end{array}\right\} = \frac{40{,}000}{1 + \frac{\text{sq. of length in inches}}{18{,}000 \times r^2}} \qquad (10)$$

in which r denotes the radius of gyration.

A column is *square bearing* when it has square ends which butt against, or are firmly connected with, an immovable surface, such as the floor of a building, or riveted connections : it is *pin and square bearing* when one end only is square bearing, and the other end presses against a close-fitting pin ; and it is *pin bearing* when both ends are thus pin-jointed with the axis of the pins in parallel directions (for example, the posts in pin-connected trusses).

To shorten the process of computation by this formula, Table VI. has been computed, which gives the ultimate strength *per square inch of cross-section* for different proportions of the length *in feet*, divided by the radius of gyration.

The radius of gyration of the principal patterns of rolled bars now on the market may be obtained from the tables given in Chapter XIII.

To use these tables, it is only necessary to divide the length of the strut (in feet) by the least radius of gyration, if the strut is free to bend either way, and from the table find the load per square inch corresponding to this ratio. The area of the cross-section, multiplied by the load, taken from the table, will give the ultimate strength of the strut or column. To find the safe load, divide by 4 for columns used in buildings, and 5 for trusses.

EXAMPLE 1.—What is the greatest safe load of a pair of Carnegie angles, 6″ × 6″, 33 pounds per foot, riveted together, 12 feet long, with square or fixed ends ; the angles being used as a strut bar in a truss ?

Ans. The least radius of gyration is 1.85, which is contained in 12, 6.5 times. The strength for a column, with square ends, for this ratio of $\frac{l}{r}$ is, from Table VI., about 34,200 pounds per square inch ; this, divided by 5, gives a safe strength of 6,840 pounds per square inch, or a total safe load for the two angles of (6,840 × 19.90) 136,116 pounds, or 68 tons.

When two or more angles, channels, or I-beams are connected together by lattice work, the radius of gyration for the whole section should first be obtained, and then the method of calculation is the same as for a single bar.

Channel bars are generally used in pairs, either connected by lattice work, or, where additional strength is required, by wrought-iron

plates riveted to the flanges of the channels. In such cases, the channels should be spaced far enough apart so that the column will be weakest in the direction of the web; *i.e.*, with neutral axis at right angles to the web, for which case the radius of gyration of the column is the same as that of a single channel.

In Table VII. the quantities d and D show the distance that the channels should be separated to have the same radius of gyration about either axis.

If the radius of gyration is wanted for the neutral axis through the centre of section parallel with web, it can readily be found, as the distance between the centre of gravity of channel and centre of section with the aid of Column VI., in tables, pages 301–21.

If two channels are connected by means of two plates, instead of lattice bars, it is necessary to obtain, first, the moment of inertia of the section, whence the radius of gyration is found as the square root of the quotient of the moment of inertia divided by the area of the section.

This moment of inertia, for a neutral axis, through centre of section perpendicular to the plates, is equal to the cube of the width of the plate, multiplied by $\frac{1}{12}$ of the thickness of the two plates added, plus the combined area of the two channels multiplied by the square of the distance from their centres of gravity to the neutral axis. For a neutral axis in a direction parallel to the plates, it is equal to the moments of inertia of the channels as found in the tables, increased by the area of the two plates multiplied by the square of the distance between the centre of the plate and the centre of the section.

The strength of such a strut may, however, be calculated with sufficient accuracy for most purposes, by taking the radius of gyration of a single channel, and getting the strength per square inch of cross-section, and then multiplying by the total area of the section. If the channels are spaced according to Table VII., or even greater, the true radius of gyration will be a little larger than that of the single channel, so that what error there is will be on the safe side.

Table VII. has been computed on this basis, giving the strength of two channels, used as a strut. The heavy figures give the safe load (factor of safety of 5) for the two channels latticed together, and the figures in italics give the safe load per square inch of section; so that, in case the pair of channels alone do not give sufficient strength, one can readily tell how much additional area will be required. Table VIII. gives the safe load of Carnegie T-bars, used singly.

EXAMPLE 2.—A certain strut in a roof truss (18 feet long) has to withstand a stress of 50 tons, and it is desired to use two channels for the purpose; what sized channels will be required, the strut having pin joints?

Ans. Looking down the column headed 18 (Table VII.), we find the nearest load under 50 tons is 40.8, for two 10″ channels, pin bearing, and the safe strength per square inch is 3.4 tons. As the load in the table lacks 9.2 tons of that required, the section of the channels must be increased by $\frac{9.2}{3.4}$, or 2.7 square inches, which is equivalent to 9 pounds per foot additional weight for the two channels; so that we should use two 10″ channels, weighing $24\frac{1}{2}$ pounds per foot each, and the channels should be spaced 9.1″ out to out, the flanges being turned in.

In pin-connected trusses, two channels make the most practical form of strut bar.

A common form of column or strut to be recommended for comparatively light loads is that formed simply of two angles riveted together back to back, or four angles united either with a single course of lattice bars or a central web plate, as in Fig. 4, page 264.

The radii of gyration for such struts are tabulated on pages 319–21.

In cases where four angles are used, the two pairs should be spaced far enough apart to make the column weakest about a neutral axis parallel to the central web or latticing. The radius of gyration will then be the same as that given in the tables for a single pair of angles, since the moment of inertia of the web plate about such an axis is so small that it may be disregarded entirely.

EXAMPLE 3.—A strut 16 feet long, to be fixed rigidly at both ends, is needed for supporting a load of 80,000 pounds. It is to be composed of two pairs of angles, united with a single line of $\frac{1}{4}$″ lattice bars along the central plane. What weight of angles will be required, with a safety factor of 5?

Ans. We will assume four 3″ × 4″ angles, and determine the thickness of metal required. The angles must be spread $\frac{1}{2}$″ in order to admit the latticing. From the table on page 321, we find the radius of gyration of a pair of light 3″ × 4″ angles with the 3″ legs parallel and $\frac{1}{2}$″ apart to be 1.97″. Hence the value of $\frac{l}{r} = \frac{16}{1.97} = 8.1$, for which the ultimate strength, as per Table VI. = 31,680 pounds.

The allowable strain per square inch with a safety factor of 5 will therefore be $31,680 \div 5 = 6,340$ pounds, and the area of the required cross-section $80,000 \div 6,340 = 12.62$ square inches, or 3.16 square inches for each angle. Hence the weight per foot of each

TABLE VI.

Ultimate Strength of Wrought-iron Columns.

For different proportions of length in feet (= l)
To least radius of gyration in inches (= r).

To obtain Safe Resistance :

For quiescent loads, as in buildings, divide by 4.
For moving loads, as in bridges, divide by 5.

$\frac{l}{r}$	Ultimate strength in pounds per square inch.		
	Square.	Pin and square.	Pin.
3.0	38,610	37,950	37,210
3.2	38,430	37,680	36,970
3.4	38,230	37,400	36,610
3.6	38,030	37,110	36,240
3.8	37,820	36,810	35,860
4.0	37,590	36,500	35,460
4.2	37,360	36,170	35,050
4.4	37,120	35,840	34,640
4.6	36,870	35,500	34,210
4.8	36,620	35,140	33,770
5.0	36,360	34,780	33,330
5.2	36,090	34,420	32,890
5.4	35,820	34,050	32,440
5.6	35,540	33,670	31,980
5.8	35,260	33,280	31,520
6.0	34,970	32,890	31,060
6.2	34,670	32,500	30,590
6.4	34,370	32,110	30,130
6.6	34,060	31,710	29,670
6.8	33,750	31,310	29,200
7.0	33,440	30,910	28,740
7.2	33,130	30,510	28,270
7.4	32,810	30,110	27,820
7.6	32,490	29,710	27,360
7.8	32,170	29,310	26,910
8.0	31,850	28,900	26,460
8.2	31,520	28,500	26,010
8.4	31,190	28,100	25,570
8.6	30,870	27,700	25,130
8.8	30,540	27,310	24,700
9.0	30,210	26,920	24,270
9.2	29,880	26,530	23,850
9.4	29,550	26,140	23,430
9.6	29,230	25,760	23,030
9.8	28,900	25,370	22,620
10.0	28,570	25,000	22,250
10.2	28,250	24,630	21,850
10.4	27,920	24,260	21,440
10.6	27,600	23,890	21,060
10.8	27,270	23,530	20,680

$\frac{l}{r}$	Ultimate strength in pounds per square inch.		
	Square.	Pin and square.	Pin.
11.0	26,950	23,170	20,230
11.2	26,640	22,820	19,960
11.4	26,320	22,470	19,610
11.6	26,000	22,130	19,270
11.8	25,690	21,800	18,930
12.0	25,380	21,460	18,590
12.2	25,070	21,130	18,250
12.4	24,770	20,810	17,940
12.6	24,470	20,490	17,620
12.8	24,170	20,180	17,310
13.0	23,870	19,860	17,000
13.2	23,570	19,560	16,710
13.5	23,140	19,110	16,290
13.8	22,700	18,670	15,850
14.0	22,420	18,380	15,580
14.2	22,150	18,100	15,310
14.5	21,740	17,690	14,920
14.8	21,320	17,290	14,530
15.0	21,050	17,020	14,290
15.2	20,790	16,760	14,040
15.5	20,290	16,390	13,690
15.8	20,020	16,010	13,350
16.0	19,770	15,770	13,120
16.2	19,510	15,540	12,910
16.5	19,150	15,190	12,590
16.8	18,750	14,850	12,290
17.0	18,550	14,630	12,080
17.2	18,320	14,410	11,840
17.5	17,980	14,100	11,590
17.8	17,640	13,790	11,320
18.0	17,420	13,590	11,140
18.2	17,200	13,390	10,960
18.5	16,880	13,100	10,700
18.8	16,570	12,820	10,450
19.0	16,370	12,630	10,290
19.2	16,170	12,470	10,130
19.5	15,870	12,190	9,890
19.8	15,570	11,930	9,670
20.0	15,380	11,760	9,530
20.2	15,230	11,600	9,380
20.5	14,920	11,360	9,170
20.8	14,650	11,190	8,970

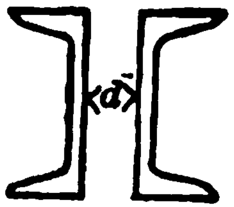
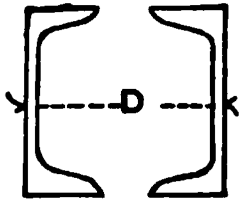


TABLE VII.



Strength of Carnegie Iron Channels, used in Pairs as Struts. *See Explanation on p.* 259.

SAFE LOAD IN TONS, FOR TWO CHANNELS.

Size of channel.	Area of section. Sq. in.	Length of strut in feet. 6	10	12	14	16	18	20	22	24	Bearing.
15″. 40 lbs.		96	96	96	96	92	90	90	88	84	Square.
R = 5.48	12	4	4	4	*4*	*3.9*	*3.8*	*3.8*	*3.7*	*3.5*	
d = 9.32		96	96	96	96	91	88	86	84.9	83	Pin.
D = 12.6		*4*	*4*	*4*	*4*	*3.8*	*3.7*	*3.6*	*3.54*	*3.46*	
12″. 30 lbs.		72	72	72	68	68	67	66	64.8	6.37	Square.
R = 4.23	9	*4*	*4*	*4*	*3.8*	*3.78*	*3.73*	*3.67*	*3.6*	*3.54*	
d = 7.2		72	72	70.2	66.2	64.4	63	61.2	59.2	57.2	Pin.
D = 9.72		*4*	*4*	*3.9*	*3.68*	*3.58*	*3.5*	*3.4*	*3.29*	*3.18*	
10″. 20 lbs.		48	48	46.2	45.6	44.8	44.04	43.2	42.3	41.5	Square.
R = 3.85	6	*4*	*4*	*3.85*	*3.8*	*3.74*	*3.67*	*3.6*	*3.53*	*3.46*	
d = 6.3		48	46.8	44.4	43.4	42	40.8	39.4	38.04	36.3	Pin.
D = 9.1		*4*	*3.9*	*3.7*	*3.62*	*3.5*	*3.4*	*3.29*	*3.17*	*3.03*	
9″. 16 lbs.		38.4	38.4	36.48	36	35.3	34.5	33.8	32.9	32	Square.
R = 3.43	4.8	*4*	*4*	*3.8*	*3.75*	*3.68*	*3.6*	*3.52*	*3.43*	*3.34*	
d = 5.6		38.4	36.6	34.9	33.8	32.6	31.58	30.24	28.9	27.55	Pin.
D = 8.12		*4*	*3.82*	*3.64*	*3.52*	*3.4*	*3.29*	*3.15*	*3.01*	*2.87*	
8″. 16 lbs.		38.4	36.48	35.8	35.04	34.08	33.1	32	30.9	29.95	Square.
R = 2.85	4.8	*4*	*3.8*	*3.73*	*3.65*	*3.55*	*3.45*	*3.34*	*3.22*	*3.12*	
d = 4.56		37.44	34.9	33.6	32.16	30.7	29.08	27.55	26.01	24.28	Pin.
D = 6.48		*3.9*	*3.64*	*3.5*	*3.35*	*3.2*	*3.03*	*2.87*	*2.71*	*2.53*	
7″. 13 lbs.		31.2	29.32	28.54	27.69	26.75	25.82	24.8	23.79	22.77	Square.
d = 4.02	3.9	*4*	*3.76*	*3.66*	*3.55*	*3.43*	*3.31*	*3.18*	*3.05*	*2.92*	
D = 5.96		28.86	27.3	26.28	24.88	23.48	21.89	20.59	19.26	17.94	Pin.
R = 2.5		*3.7*	*3.5*	*3.37*	*3.19*	*3.01*	*2.82*	*2.64*	*2.47*	*2.30*	

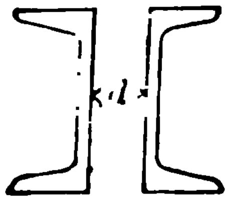
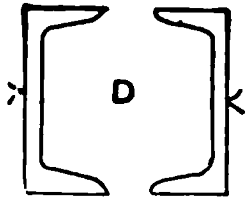


TABLE VII.—*Continued.*



Strength of Carnegie Iron Channels, as Struts.—Continued.

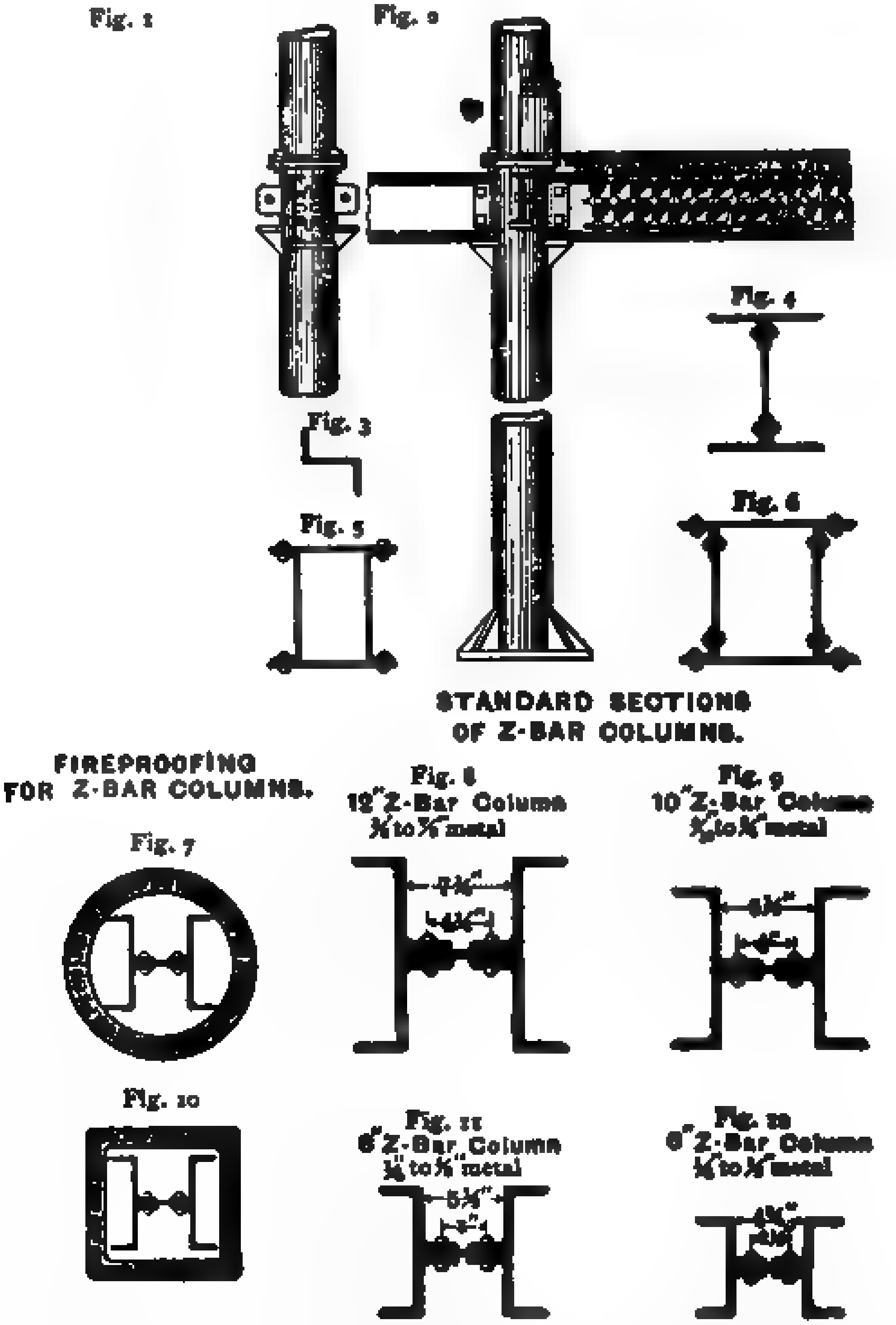
SAFE LOAD IN TONS FOR TWO CHANNELS.

Size of channel.	Area of Section. Sq. in.	Length of strut in feet. 6	8	10	12	14	16	18	20	22	Bearing.
6″. 16 lbs.		37.44	36.38	35.32	34.17	32.92	31.48	29.95	28.41	27.16	Square.
R = 2.16	4.8	*3.9*	*3.79*	*3.68*	*3.56*	*3.43*	*3.28*	*3.12*	*2.96*	*2.83*	
d = 3.06		36.48	34.75	32.83	30.72	28.8	26.7	24.57	22.65	20.92	Pin.
D = 5.58		*3.8*	*3.62*	*3.42*	*3.2*	*3*	*2.78*	*2.56*	*2.36*	*2.18*	
6″. 7½ lbs.		16.98	16.8	16.36	15.84	15.40	14.78	14.17	13.64	12.98	Square.
R = 2.34	2.2	*3.86*	*3.82*	*3.72*	*3.6*	*3.5*	*3.36*	*3.22*	*3.1*	*2.95*	
d = 3.72		16.41	16.1	15.4	14.52	13.64	12.76	11.92	11.17	10.12	Pin.
D = 5.64		*3.73*	*3.66*	*3.5*	*3.3*	*3.1*	*2.9*	*2.71*	*2.59*	*2.3*	
5″. 14 lbs.		32.08	31.08	29.73	28.3	26.88	25.36	23.68	22.17	20.74	Square
R = 1.77	4.2	*3.82*	*3.7*	*3.54*	*3.37*	*3.2*	*3.02*	*2.82*	*2.64*	*2.47*	
d = 2.32		30.74	28.89	26.71	24.52	22.34	20.33	18.31	16.63	15.12	Pin.
D = 4.76		*3.66*	*3.44*	*3.18*	*2.92*	*2.66*	*2.42*	*2.18*	*1.98*	*1.8*	
5″. 6 lbs.		13.89	13.54	13.07	12.6	12.02	11.44	10.87	10.3	9.72	Square.
R = 1.99	1.8	*3.86*	*3.76*	*3.63*	*3.5*	*3.34*	*3.18*	*3.02*	*2.86*	*2.7*	
d = 2.98		13.42	12.74	11.90	11.16	10.34	9.54	8.71	7.99	7.3	Pin.
D = 4.98		*3.73*	*3.54*	*3.33*	*3.1*	*2.87*	*2.65*	*2.42*	*2.22*	*2.03*	
4″. 9 lbs.		20.2	19.22	18.09	16.96	15.66	14.36	13.17	12.09	....	Square.
R = 1.46	2.7	*3.74*	*3.56*	*3.35*	*3.14*	*2.9*	*2.66*	*2.44*	*2.24*	....	
d = 1.8		19	17.39	16.84	15.66	13.93	12.31	10.8	8.42	....	Pin.
D = 4.04		*3.52*	*3.22*	*3.12*	*2.9*	*2.58*	*2.28*	*2*	*1.56*	...	
4″ 5 lbs.		11.84	10.89	10.35	9.78	9.15	8.58	7.98	7.38	....	Square.
R = 1.57	1.5	*3.78*	*3.63*	*3.45*	*3.26*	*3.05*	*2.86*	*2.66*	*2.46*	...	
d = 2.24		10.74	9.99	9.09	8.25	7.47	6.66	6	5.34	....	Pin.
D = 4.04		*3.58*	*3.33*	*3.03*	*2.75*	*2.49*	*2.22*	*2*	*1.78*	....	

SAFE LOAD IN TONS; SQUARE OR FIXED ENDS.

Size flange by stem.	Weight per foot.	Area of section. Sq. in.	Least radius of gyration.	Length of strut in feet.								
				4	5	6	7	8	9	10	11	12
5 × 3	12.7	3.81	0.76	13.71 *3.6*	12.95 *3.5*	12.19 *3.2*	11.39 *2.99*	10.59 *2.78*	9.79 *2.57*	8.99 *2.36*	8.30 *2.18*	7.62 *2.00*
5 × 2½	11.9	3.57	0.64	12.35 *3.46*	11.49 *3.22*	10.56 *2.96*	9.70 *2.72*	8.81 *2.47*	8.00 *2.24*	7.21 *2.02*	6.60 *1.85*	6.00 *1.68*
4 × 5	15.2	4.56	0.79	16.50 *3.62*	15.68 *3.44*	14.77 *3.24*	13.86 *3.04*	12.95 *2.84*	12.00 *2.63*	11.12 *2.44*	10.26 *2.25*	9.44 *2.07*
4 × 5	11.8	3.54	0.78	12.80 *4.62*	12.17 *3.44*	11.47 *3.24*	10.76 *3.04*	10.05 *2.84*	9.3 *2.63*	8.64 *2.44*	7.96 *2.25*	7.32 *2.07*
4 × 4	13.4	4.02	0.84	14.71 *3.66*	14.07 *3.50*	13.35 *3.32*	12.62 *3.14*	11.82 *2.94*	10.75 *2.75*	10.21 *2.54*	9.57 *2.38*	8.84 *2.20*
4 × 3	9.1	2.73	0.86	10.05 *3.68*	9.61 *3.52*	8.92 *3.34*	8.62 *3.16*	8.08 *2.96*	7.59 *2.78*	7.10 *2.60*	6.58 *2.41*	6.11 *2.24*
3½ × 4	9.7	2.91	0.70	10.30 *3.54*	9.34 *3.21*	8.99 *3.09*	8.32 *2.86*	7.65 *2.63*	7.04 *2.42*	6.40 *2.20*	5.88 *2.02*	5.35 *1.84*
3½ × 3	8.3	2.49	0.75	8.96 *3.6*	8.47 *3.4*	7.97 *3.2*	7.44 *2.99*	6.92 *2.78*	6.40 *2.57*	5.83 *2.34*	5.43 *2.18*	5.00 *2.00*
3 × 4	11.6	3.48	0.59	11.73 *3.37*	10.82 *3.11*	9.88 *3.84*	8.94 *2.57*	8.04 *2.31*	7.24 *2.08*	6.44 *1.85*	5.78 *1.66*	5.22 *1.5*
3 × 3	6.5	1.95	0.62	6.67 *3.42*	6.20 *3.18*	5.69 *2.93*	5.17 *2.65*	4.70 *2.41*	4.23 *2.51*	3.84 *1.97*	3.43 *1.76*	3.08 *1.58*
3 × 2½	6.0	1.80	0.65	6.22 *3.46*	5.8 *3.22*	5.38 *2.96*	4.9 *2.72*	4.44 *2.47*	4.03 *2.24*	3.64 *2.02*	3.30 *1.85*	3.02 *1.68*
2½ × 3	6.0	1.80	0.51	5.76 *3.2*	5.20 *2.89*	4.68 *2.6*	4.14 *2.3*	3.6 *2.00*	3.17 *1.76*	2.84 *1.58*	....	....

angle will be 3.16 ÷ 0.3 = 10.5 lbs. This weight will be found to agree with a thickness of ½ inch for a 4″ × 3″ angle.



Failure of Columns by Deflection or Buckling.—Steel and wrought-iron columns fail either by deflecting bodily out of a straight line, or by the buckling of the metal between rivets or other points of support. Both actions may take place at the

same time, but if the latter occurs alone, it may be an indication that the rivet spacing or the thickness of the metal is insufficient.

The rule has been deduced from actual experiments upon wrought-iron columns, that the distance between centres of rivets should not exceed, in the line of strain, sixteen times the thickness of metal of the parts joined, and that the distance between rivets or other points of support, at right angles to the line of strain, should not exceed thirty-two times the thickness of the metal.

On page 264 sections are shown of some of the most common forms of steel and wrought-iron columns. Figs. 5 and 6, as well as the Phœnix and Keystone Columns illustrated on pages 267 and 277, belong to the type known as Closed Columns. As it is impracticable to repaint the inner surfaces of such columns, they should preferably be used only for interior work, where the changes in temperature are not considerable, and the air is comparatively dry. In places exposed to the extremes of temperature and unprotected from the rain, the paint on the inner surface of the column will, sooner or later, cease to be a protection to the iron, corrosion will set in, and, once begun, will continue as long as there is unoxidized metal left in the column.

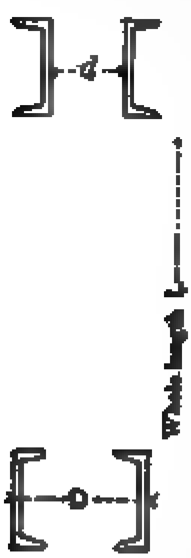


Fig 1.

Figures 4 and 8 on page 264 represent types of columns with open sections, which readily admit of repainting, and are therefore suitable for outdoor work.

Of these, the latter, designed by C. L. Strobel, C.E., and known as the Z-bar Column, is believed to offer advantages equal, if not superior to those of any other steel or wrought-iron column in the market.

Bracing of Channels.—When channels are connected by lattice work (as in Fig. 1), that there may not be a tendency in the channels to bend between the points of bracing, the distance l should be made to equal the total length of strut, multiplied by the least radius of gyration of a single column, and the product divided by the least radius of gyration for the whole section ; or, $l = \frac{r\,L}{R}$ where the letters have the following significance :

l = length between bracing.
L = total length of strut.
r = least radius of gyration for a single channel.
R = least radius of gyration for the whole section.

When the radius of gyration of channels, about an axis parallel with the web, is not given, it will be sufficiently accurate to use for r the distance given in Column VI. in the tables on pages 301-321.

EXAMPLE 4.—We will determine the distance l, for the strut calculated in Example 2. In this case $L = 18$ feet, or 216 inches, $R = 3.85$; and in Column VI., page 304, the distance d for a 20-pound channel is .70, for a 35-pound channel .75, so that we will assume .72 as the proper distance for a 24-pound channel ; or $r = .72$; then $l = \frac{216 \times .72}{3.85} = 40$ inches. This same rule will also apply for angles, though with them the lattice work is generally doubled, as in Fig. 2.

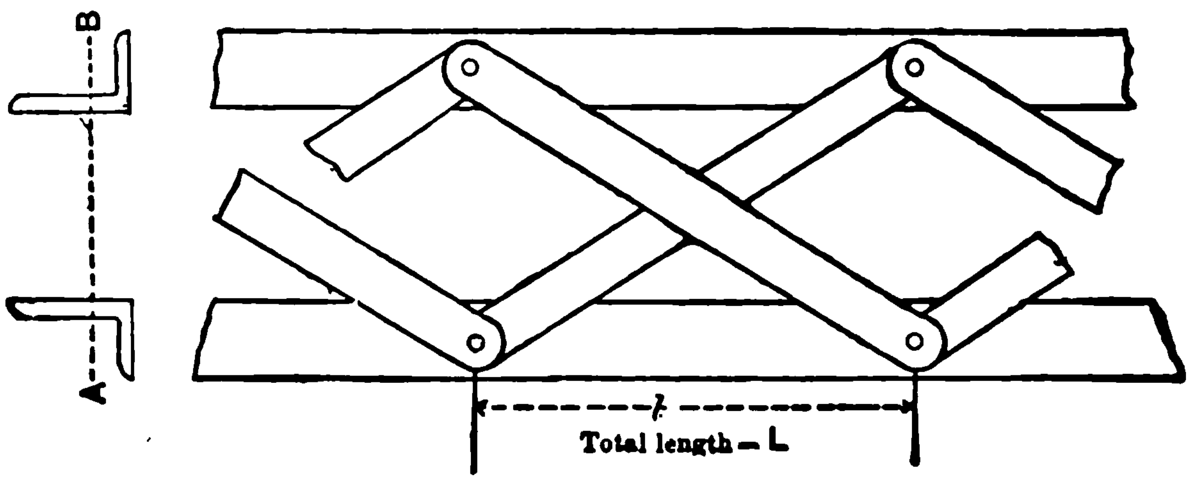


FIG. 2.

Steel Columns.

"Experiments thus far made upon steel struts indicate that for lengths up to 90 radii of gyration," (or 7.5 in Table VI.,) "their ultimate strength is about 20 per cent. higher than for iron. Beyond this point, the excess of strength diminishes until it becomes zero at about 200 radii. After passing this limit, the compressive resistance of steel and iron seems to become practically equal."* In Tables VII. and VIII. the loads to the left of heavy black line are for ratios less than 90 diameters, and those to the right for ratios above that limit.

Special Forms of Wrought-iron and Steel Columns.

The Phœnix Segmental Column † has now been on the market for a number of years, and is very extensively used in buildings, and also for posts in bridges.

* C. L. Strobel, C.E.

† Manufactured by the Phœnix Iron Company, Philadelphia.

tages are : Economy of metal, simplicity of construc-
bility to the requirements of building construction, and
s.

umns are made up of the rolled segments "C," which are riveted together, by rivets about six inches apart, by means of flanges along their sides, as shown at "A" (Fig. 13). Between every two segments an iron bar is frequently inserted, through which the rivets pass. These bars, or "flats" as they are called, increase the area of the cross-section, and contribute much to the strength of the pillar. Table IX. gives the sizes of the columns rolled by the Phœnix Iron Company, as published in their book of sections.

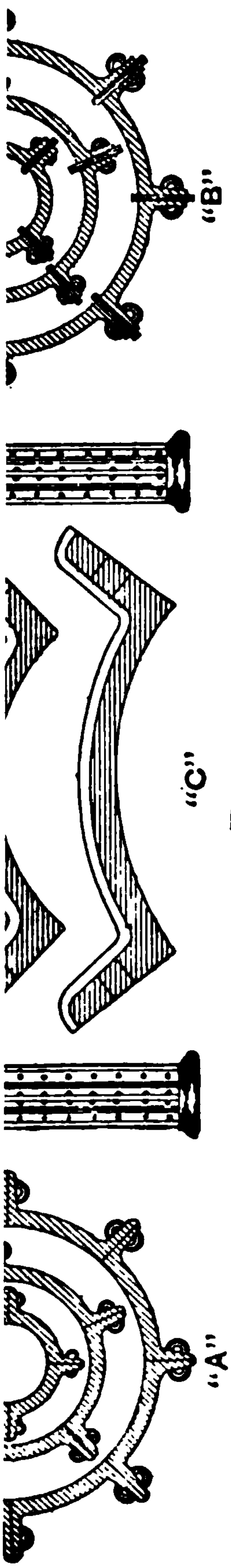


FIG. 13.

The interior surfaces of all Phœnix columns are thoroughly painted before riveting the segments together. After twenty years of service in exposed situations, columns have been cut open and found uninjured by rust, and the paint still in good condition.

The illustrations on pages 270 and 271 show methods of joining the several tiers of columns in a building, and the connections with girders, etc.

Bearings for girders or beams at irregular heights are provided by projecting brackets that are properly riveted to a segment, or by a plate passing transversely through the column between the flanges, with seating angles along its upper edge.

For joining columns at the levels of different tiers, inside sleeves of wrought iron may be used. They are riveted to the segments of the lower column, and
cting tenon which is fastened by diagonal through bolts
column when it is put in place.

ine the actual value of Phœnix columns under loads,
have been made at different times and on various
d especially that of the United States Government at

TABLE VI.

Ultimate Strength of Wrought-iron Columns.

For different proportions of length in feet ($= l$)
To least radius of gyration in inches ($= r$).

To obtain Safe Resistance :

For quiescent loads, as in buildings, divide by 4.
For

[illegible]

	22,150	18,100	[illegible]
6.0	21,740	17,690	14,920
6.2	21,390	17,390	14,580
6.4	21,050	17,020	14,290
6.6	20,790	16,760	14,040
6.8	20,390	16,390	13,690
7.0	20,020	16,010	13,350
7.2	19,760	15,770	13,190
7.4	19,510	15,540	12,910
7.6	19,150	15,190	12,590
7.8	18,730	14,850	12,330
8.0	18,350	14,690	12,090
8.2	18,020	14,410	11,880
8.4	17,830	14,100	11,590
8.6	17,640	13,720	11,330
8.8	17,420	13,590	11,140
	17,200	13,390	10,960
9.0	16,890	13,100	10,700
9.2	16,570	12,830	10,450
9.4	16,370	12,680	10,390
9.6	16,170	12,470	[illegible]
9.8	15,870	12,190	[illegible]
10.0	15,570	11,980	9,670
10.2	15,390	11,760	9,590
10.4	15,200	11,600	9,380
10.6	14,930	11,360	9,170
10.8	14,650	11,120	8,970

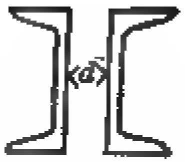
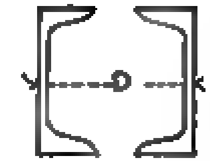


TABLE VII.

Strength of Carnegie Iron Channels, used in Pairs as Struts. *See Explanation on p.* 259.



SAFE LOAD IN TONS, FOR TWO CHANNELS.

TABLE IX.

Sizes of Phœnix Columns.

Mark.	One Segment.		One Column.		Least radius of gyration in inches.
	Thickness in inches.	Weight in pounds per yard.	Area in sq. inches.	Weight in pounds per foot.	
A 4 segment. 3⅝" inter. diam.	3/16	9½	3.8	12.6	1.45
	1/4	12	4.8	16.0	1.50
	5/16	14½	5.8	19.3	1.55
	3/8	17	6.8	22.6	1.59
B¹ 4 segment. 4 13/16" inter. diam.	1/4	16	6.4	21.3	1.92
	5/16	19½	7.8	26.0	1.96
	3/8	23	9.2	30.6	2.02
	7/16	26½	10.6	35.3	2.07
	1/2	30	12.0	40.0	2.11
	9/16	33½	13.4	44.6	2.16
	5/8	37	14.8	49.3	2.20
B² 4 segment. 5 15/16" inter. diam.	1/4	18½	7.4	24.6	2.34
	5/16	22½	9.0	30.0	2.39
	3/8	26½	10.6	35.3	2.43
	7/16	30½	12.2	40.6	2.48
	1/2	34½	13.8	46.0	2.52
	9/16	38½	15.4	51.3	2.57
	5/8	42½	17.0	56.6	2.61
C 4 segment. 7 3/16" inter. diam.	1/4	25	10.0	33.3	2.80
	5/16	30	12.0	40.0	2.85
	3/8	35	14.0	46.6	2.90
	7/16	40	16.0	53.3	2.94
	1/2	45	18.0	60.0	2.98
	9/16	48	19.2	64.0	3.03
	5/8	53	21.2	70.6	3.08
	11/16	58	23.2	77.3	3.12
	3/4	63	25.2	84.0	3.16
	13/16	68	27.2	90.6	3.21
	7/8	73	29.2	97.3	3.26
	1	83	33.2	110.6	3.34
	1⅛	93	37.2	124.0	3.43
	1¼	103	41.2	137.3	3.52

TABLE IX.—*Concluded.*

Sizes of Phœnix Columns.

NOTE.—The weight of rivet-heads adds from two to five per cent. to the weight of finished columns.

the Watertown (Mass.) Arsenal. From these experiments formulas have been deduced from which the accompanying tables have been prepared, in which are shown the safe loads in net tons for each size and length of the several patterns made.

columns are unequally loaded, then it will be advisable t
the tabular figures or use heavier sections for the case, a
indicated by the circumstances.

Steel Columns.—These tables have been prepared
columns. If it is desired to use steel, it will be proper to
for loads from 15 to 20 per cent. more than those give
tables, the greater value being for short, and the lesser
columns.

SAFE LOADS IN TONS OF 2,000 POUNDS.

PHŒNIX IRON COLUMNS.

Square Ends.

---

4 Segment, A Column, $3\frac{5}{8}''$ Inside Diameter.

Length of column in feet.	$\frac{3}{16}''$ 12.6 lbs. per ft. 3.8 □ in.	$\frac{1}{4}''$ 16 lbs. per ft. 4.8 □ in.	$\frac{5}{16}''$ 19.3 lbs. per ft. 5.8 □ in.	$\frac{3}{8}''$ 22.6 lbs. per ft. 6.8 □ in.
10	17.29	22.17	27.20	32.36
12	16.87	21.65	26.57	31.63
14	15.99	20.54	25.23	30.05
16	15.08	19.39	23.84	28.43
18	14.17	18.24	22.45	26.79
20	13.29	17.12	21.10	25.21
22	12.39	15.99	19.73	23.61
24	11.57	14.95	18.47	22.13

4 Segment, B[1] Column, $4\frac{13}{16}''$ Inside Diameter.

Length of column in feet.	$\frac{1}{4}''$ 21.3 lbs. per ft. 6.4 □ in.	$\frac{5}{16}''$ 26 lbs. per ft. 7.8 □ in.	$\frac{3}{8}''$ 30.6 lbs. per ft. 9.2 □ in.	$\frac{7}{16}''$ 35.3 lbs. per ft. 10.6 □ in.	$\frac{1}{2}''$ 40 lbs. per ft. 12 □ in.	$\frac{9}{16}''$ 44.6 lbs. per ft. 13.4 □ in.	$\frac{5}{8}''$ 49.3 lbs. per ft. 14.8 □ in.
10	30.30	37.40	44.67	52.10	59.71	67.47	75.41
12	29.45	36.36	43.44	50.68	58.10	65.68	73.43
14	28.49	35.20	42.07	49.10	56.31	63.69	71.23
16	27.46	33.94	40.59	47.40	54.38	61.53	68.84
18	26.40	32.64	39.05	45.63	52.38	59.29	66.37
20	25.28	31.27	37.44	43.77	50.28	56.95	63.78
22	24.14	29.89	35.80	41.90	48.15	54.57	61.16
24	23.00	28.50	34.17	40.01	46.02	52.19	58.53
26	21.88	27.14	32.56	38.16	43.92	49.84	55.94

4 Segment, B[2] Column, $5\frac{15}{16}''$ Inside Diameter.

Length of column in feet.	$\frac{1}{4}''$ 24.6 lbs. per ft. 7.4 □ in.	$\frac{5}{16}''$ 30 lbs. per ft. 9 □ in.	$\frac{3}{8}''$ 35.3 lbs. per ft. 10.6 □ in.	$\frac{7}{16}''$ 40.6 lbs. per ft. 12.2 □ in.	$\frac{1}{2}''$ 46 lbs. per ft. 13.8 □ in.	$\frac{9}{16}''$ 51.3 lbs. per ft. 15.4 □ in.	$\frac{5}{8}''$ 56.6 lbs. per ft. 17 □ in.
10	35.97	44.20	52.[illegible]9	61.14	60.85	78.72	87.75
12	35.25	43.33	51.56	59.96	68.51	77.23	86.10
14	34.43	42.32	50.38	58.59	66.97	75.50	84.20
16	33.[illegible]3	41.23	49.09	57.12	65.30	73.64	82.14
18	32.57	40.06	47.72	55.53	63.50	71.64	79.93
20	31.55	38.83	46.26	53.86	61.61	69.52	77.60
22	30.48	37.53	44.73	52.09	59.61	67.29	75.14
24	29.41	36.22	43.19	50.32	57.61	65.06	72.67
26	28.31	34.89	41.62	48.51	55.57	62.78	70.15
28	27.23	33.57	40.06	46.72	53.54	60.52	67.66

SAFE LOADS IN TONS OF 2,000 POUNDS.

PHŒNIX IRON COLUMNS.

Square Ends.

4 SEGMENT, C COLUMN, 7 3/16″ INSIDE DIAMETER.

Length of column in feet.	1/4″ 33.3 lbs. per ft. 10 □ in.	5/16″ 40 lbs. per ft. 12 □ in.	3/8″ 46.6 lbs. per ft. 14 □ in.	7/16″ 53.3 lbs. per ft. 16 □ in.	1/2″ 60 lbs. per ft. 18 □ in.	9/16″ 64 lbs. per ft. 19.2 □ in.	5/8″ 70.6 lbs. per ft. 21.2 □ in.
10	50.97	61.16	71.35	81.55	91.74	97.86	108.05
12	50.33	60.40	70.46	80.53	90.60	96.64	106.71
14	49.62	59.54	69.46	79.39	89.31	95.27	105.19
16	48.91	58.59	68.48	78.26	88.04	93.91	103.69
18	47.87	57.45	67.02	76.60	86.17	91.92	101.49
20	46.93	56.31	65.70	75.08	84.47	90.10	99.49
22	45.92	55.11	64.29	73.48	82.66	88.17	97.36
24	44.86	53.83	62.81	71.78	80.75	86.14	95.11
26	43.77	52.53	61.28	70.04	78.79	84.04	92.80
28	42.63	51.16	59.68	68.21	76.74	81.85	90.38
30	41.48	49.78	58.07	66.37	74.67	79.65	87.94
32		48.42	56.49	64.56	72.63	77.47	85.54
34			54.85	62.69	70.53	75.23	83.07
36				60.83	68.43	73.00	80.60
38					66.37	70.80	78.17
40						68.61	75.75

Length of column in feet.	11/16″ 77.3 lbs. per ft. 23.2 □ in.	3/4″ 84 lbs. per ft. 25.2 □ in.	13/16″ 90.6 lbs. per ft. 27.2 □ in.	7/8″ 97.3 lbs. per ft. 29.2 □ in.	1″ 110.6 lbs. per ft. 33.2 □ in.	1 1/8″ 124 lbs. per ft. 37.2 □ in.	1 1/4″ 137.3 lbs. per ft. 41.2 □ in.
10	118.26	128.45	138.65	148.84	169.23	189.62	210.01
12	116.77	126.84	136.91	146.97	167.11	187.24	207.38
14	115.11	125.04	134.96	144.89	164.73	184.58	204.43
16	113.48	123.26	133.04	142.83	162.39	181.96	201.52
18	111.07	120.64	130.22	139.79	158.94	178.09	197.24
20	108.87	118.26	127.64	137.03	155.80	174.57	193.35
22	106.54	115.73	124.91	134.10	152.47	170.84	189.21
24	104.08	113.05	122.03	131.01	148.95	166.89	184.84
26	101.55	110.31	119.06	127.82	145.33	162.84	180.35
28	98.91	107.44	115.96	124.49	141.54	158.60	175.65
30	96.24	104.54	112.83	121.13	137.73	154.32	170.91
32	93.61	101.68	109.75	117.82	133.91	150.10	166.24
34	90.90	98.74	106.58	114.42	130.09	145.76	161.44
36	88.20	95.81	103.41	111.01	126.22	141.43	156.64
38	85.55	92.92	100.30	107.67	122.42	137.17	151.92
40	82.90	90.05	97.19	104.34	118.64	132.93	147.22

SAFE LOADS IN TONS OF 2,000 POUNDS.

PHŒNIX IRON COLUMNS.

Square Ends.

6 SEGMENT, E COLUMN, 11″ INSIDE DIAMETER.

Length of column in feet.	1/4″ 56 lbs. per ft. 16.8 □ in.	5/16″ 64 lbs. per ft. 19.2 □ in.	3/8″ 72 lbs. per ft. 21.6 □ in.	7/16″ 80 lbs. per ft. 24 □ in.	1/2″ 88 lbs. per ft. 26.4 □ in.	9/16″ 96 lbs. per ft. 28.8 □ in.	5/8″ 106 lbs. per ft. 31.8 □ in.
10	86.94	99.36	111.78	124.20	136.62	149.04	164.56
12	86.41	98.76	111.11	123.45	135.80	148.14	163.57
14	85.79	98.05	110.31	122.56	134.82	147.08	162.40
16	85.09	97.24	109.40	121.56	133.71	145.87	161.06
18	84.30	96.34	108.38	120.43	132.47	144.51	159.56
20	83.44	95.36	107.28	119.20	131.12	143.04	157.95
22	82.52	94.31	106.09	117.88	129.67	141.46	156.20
24	81.51	93.15	104.80	116.44	128.09	139.73	154.29
26	80.47	91.96	103.46	114.96	126.45	137.95	152.32
28	79.38	90.72	102.06	113.40	124.74	136.08	150.25
30	78.23	89.41	100.59	111.76	122.94	134.12	148.09
32	77.02	88.03	99.03	110.04	121.04	132.04	145.80
34	75.76	86.59	97.41	108.24	119.06	129.88	143.41
36	74.50	85.15	95.79	106.44	117.08	127.72	141.03
38	73.21	83.67	94.13	104.59	115.05	125.51	138.58
40	71.90	82.17	92.44	102.72	112.99	123.26	136.10

Length of column in feet.	11/16″ 116 lbs. per ft. 34.8 □ in.	3/4″ 126 lbs. per ft. 37.8 □ in.	13/16″ 136 lbs. per ft. 40.8 □ in.	7/8″ 146 lbs. per ft. 43.8 □ in.	1″ 166 lbs. per ft. 49.8 □ in.	1 1/8″ 186 lbs. per ft. 55.8 □ in.	1 1/4″ 206 lbs. per ft. 61.8 □ in.
10	180.09	195.61	211.14	226.66	257.71	288.76	319.81
12	179.01	194.44	209.87	225.30	256.17	287.03	317.89
14	177.71	193.04	208.36	223.68	254.32	284.97	315.61
16	176.26	191.45	206.65	221.84	252.23	282.62	313.01
18	174.62	189.68	204.73	219.78	249.89	280.00	310.10
20	172.85	187.75	202.65	217.55	247.35	277.15	306.96
22	170.93	185.67	200.40	215.14	244.62	274.08	303.56
24	168.84	183.40	197.96	212.51	241.62	270.74	299.85
26	166.69	181.06	195.43	209.80	238.54	267.28	296.02
28	164.43	178.60	192.78	206.95	235.30	263.65	292.00
30	162.06	176.03	190.00	203.97	231.91	259.86	287.80
32	159.55	173.31	187.06	200.82	228.33	255.84	283.35
34	156.94	170.47	184.00	197.53	224.59	251.65	278.71
36	154.33	167.64	180.94	194.25	220.86	247.47	274.08
38	151.65	164.73	177.80	190.88	217.02	243.17	269.32
40	148.94	161.78	174.62	187.46	213.14	238.82	264.50

SAFE LOADS IN TONS OF 2,000 POUNDS.

PHŒNIX IRON COLUMNS.

Square Ends.

8 SEGMENT, G COLUMN, 14⅜″ INSIDE DIAMETER.

Length of column in feet.	5/16″ 80 lbs. per ft. 24 □ in.	⅜″ 93.3 lbs. per ft. 28 □ in.	7/16″ 106.6 lbs. per ft. 32 □ in.	½″ 120 lbs. per ft. 36 □ in.	9/16″ 133.3 lbs. per ft. 40 □ in.	⅝″ 146.6 lbs. per ft. 44 □ in.	11/16″ 160 lbs. per ft. 48 □ in.
10	124.92	145.74	166.56	187.38	208.20	229.02	249.84
12	124.44	145.18	165.92	186.66	207.40	228.14	248.88
14	123.91	144.56	165.21	185.86	206.52	227.17	247.82
16	123.28	143.83	164.38	184.93	205.48	226.02	246.57
18	122.59	143.02	163.45	183.88	204.32	224.75	245.18
20	121.82	142.12	162.43	182.73	203.04	223.34	243.64
22	120.98	141.14	161.31	181.47	201.64	221.80	241.96
24	120.04	140.05	160.06	180.07	200.08	220.08	240.09
26	119.11	138.96	158.81	178.66	198.52	218.37	238.22
28	118.08	137.76	157.44	177.12	196.80	216.48	236.16
30	117.00	136.50	156.00	175.50	195.00	214.50	234.00
32	115.84	135.15	154.46	173.77	193.08	212.38	231.69
34	114.64	133.75	152.86	171.97	191.08	210.18	229.29
36	113.28	132.16	151.04	169.92	188.80	207.68	226.56
38	112.08	130.76	149.44	168.12	186.80	205.48	224.16
40	110.80	129.27	147.74	166.21	184.68	203.14	221.61

Length of column in feet.	¾″ 173.3 lbs. per ft. 52 □ in.	13/16″ 186.6 lbs. per ft. 56 □ in.	⅞″ 200 lbs. per ft. 60 □ in.	1″ 226.6 lbs. per ft. 68 □ in.	⅛″ 253.3 lbs. per ft. 76 □ in.	1¼″ 280 lbs. per ft. 84 □ in.	1⅜″ 306.6 lbs. per ft. 92 □ in.
10	270.66	291.48	312.30	353.94	395.58	437.22	478.86
12	269.62	290.36	311.10	352.58	394.06	435.54	477.02
14	268.47	289.12	309.78	351.08	392.38	433.69	474.99
16	267.12	287.67	308.22	349.31	390.41	431.50	472.60
18	265.61	286.04	306.48	347.34	388.20	429.07	469.93
20	263.95	284.25	304.56	345.10	385.77	426.38	466.99
22	262.13	282.29	302.46	342.78	383.11	423.44	463.77
24	260.10	280.11	300.12	340.13	380.15	420.16	460.18
26	258.07	277.92	297.78	337.48	377.18	416.89	456.59
28	255.84	275.52	295.20	334.56	373.92	413.28	452.64
30	253.50	273.00	292.50	331.50	370.50	409.50	448.50
32	251.00	270.31	289.62	328.23	366.85	405.46	444.08
34	248.40	267.51	286.62	324.83	363.05	401.26	439.48
36	245.44	264.32	283.20	320.96	358.72	396.48	434.24
38	243.84	261.52	280.20	317.56	354.92	392.28	429.64
40	240.08	258.55	277.02	313.95	350.89	387.82	424.76

Keystone Octagon Column.

Another special form of wrought-iron column is that known as the Keystone Octagon Column, manufactured by Carnegie, Phipps & Co. It is made of four rolled segments of wrought iron, riveted together as shown in Fig. 5.

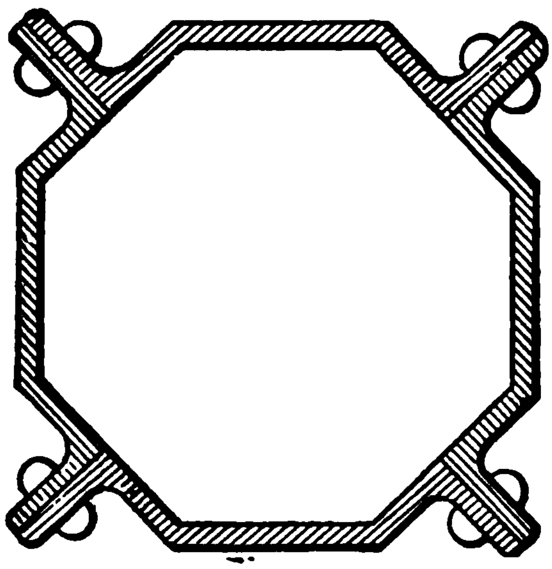


FIG. 5.

The table on the following page gives the diameters, areas, and weights of these columns as rolled. To compute the strength of these columns it is first necessary to find the radius of gyration (r), when the strength per square inch can then be determined from Table VI.

The radius of gyration may be found by the following formulæ :

$$I = \frac{D^4 - d^4}{12}; \qquad r = \sqrt{\frac{I}{A}}, \text{ in which}$$

$I =$ moment of inertia ; $D =$ outside diameter ;
$A =$ area of column ; $d =$ inside diameter ;
$r =$ radius of gyration.

KEYSTONE OCTAGON COLUMNS.

THICKNESSES AND CORRESPONDING AREAS AND WEIGHTS PER FOOT.

Thickness.	10-Inch Column.			8-Inch Column.			6-Inch Column.			4-Inch Column.			Thickness.
	4 Segments.		Weight of one segment.	4 Segments.		Weight of one segment.	4 Segments.		Weight of one segment.	4 Segments.		Weight of one segment.	
	Area.	Weight.		Area.	Weight.		Area.	Weight.		Area.	Weight.		
Inch.	sq. in.	lbs.	lbs.	sq. in.	lbs.	lbs.	sq. in.	lbs.	lbs.	sq. in.	lbs.	lbs.	Inch.
$\frac{3}{16}$	-	-	-	-	-	-	5.60	18.7	4.7	3.91	13.0	3.3	$\frac{3}{16}$
$\frac{1}{4}$	-	-	-	9.78	32.6	8.2	7.13	23.8	5.9	4.98	16.6	4.2	$\frac{1}{4}$
$\frac{5}{16}$	14.22	47.4	11.9	11.80	39.3	9.8	8.66	28.9	7.2	6.05	20.2	5.0	$\frac{5}{16}$
$\frac{3}{8}$	16.58	55.3	13.8	13.81	46.0	11.5	10.20	34.0	8.5	7.12	23.7	5.9	$\frac{3}{8}$
$\frac{7}{16}$	18.94	63.1	15.8	15.83	52.8	13 2	11.73	39.1	9.8	8.20	27.3	6.8	$\frac{7}{16}$
$\frac{1}{2}$	21.30	71.0	17.8	17.85	59.5	14.9	13.26	44.2	11.1	9.27	30.9	7.7	$\frac{1}{2}$
$\frac{9}{16}$	23.66	78.9	10.7	19.86	66.2	16.6	14.79	49.3	12.3	-	-	-	$\frac{9}{16}$
$\frac{5}{8}$	26.01	86.7	21.7	21.88	72.9	18.2	16.32	54.4	13.6	-	-	-	$\frac{5}{8}$
$\frac{11}{16}$	28.37	94.6	23.6	23.80	79.6	19.9	-	-	-	-	-	-	-
$\frac{3}{4}$	30.73	102.4	25.6	25.91	86.4	21.6	-	-	-	-	-	-	-
$\frac{13}{16}$	33.09	110.3	27.6	-	-	-	-	-	-	-	-	-	-
$\frac{7}{8}$	35.45	118.2	20.5	-	-	-	-	-	-	-	-	-	-

Z-Bar Columns.

Within the past three years, what is known as the Z-bar column has been introduced, and is now manufactured by all the leading iron mills. It is built up of four Z-bars, riveted together, as shown in Figs. 7 to 12, page 264.

The dimensions of the different shapes manufactured will be found in the tables given in Chapter XIII.

This column possesses so many advantages for building purposes that it is undoubtedly destined to be extensively used.

Its claims for superiority are based mainly on the following qualities:

1. *Cheapness.*—The Z-bars are furnished at a lower price per pound than channels and I-beams, and only two rows of rivets are required, while four or more are used for any other column of an equal sectional area.

2. *High Ultimate Resistance to Compression.*—Careful tests made upon fifteen full-sized (Carnegie) specimens, in which the web plates were replaced by lattice bars, showed an average ultimate resistance per square inch of 35,650 pounds for lengths ranging from 64 to 88 radii. These results are as favorable as have been obtained for closed cylindrical columns, and are more favorable than have been obtained for any other open columns. For detailed report of the tests referred to, see paper by C. L. Strobel, in *Trans. Am. Soc. C. E.*, April, 1888.

3. *Great Adaptability for Effecting Connections with I-beams.*—When used in buildings, for supporting single floor beams, or double beam girders, this quality is of the greatest importance. The illustrations on pages 280 and 281 show different methods of making the connections, as employed by Carnegie, Phipps & Co. This column may be easily covered with terra-cotta blocks, for fireproofing, and finishing with plaster or cement, and the air-space between the tiling and the metal adds to the protection of the latter in the event of fire. The recesses in the columns may be used to good advantage for conducting water and gas pipes, electric wires, etc.

4. *Favorable Form for Inspection and Repairing.*—This is a very desirable feature when used for out-door work.

When unusually heavy loads must be provided for, as in the case of columns for the lower stories of very high buildings, the standard sections of Z-bar columns may be reënforced to the required strength by using either a double central web plate, or by the addition of outside cover plates, or, if need be, both, forming thus a

DETAILS OF STANDARD CONNECTIONS OF I BEAMS TO Z-BAR COLUMNS.

Connections of a single I Beam to Flanges of Z Bars.

Fig. 1 Fig. 2 Fig. 3 Fig. 4

20" I Beams
44 Tons.

15" and 12" I Beams
35 Tons.

10½", 10," 9" and 8"
I Beams
17.6 Tons.

7" and 6"
I Beams
8.8 Tons.

Connections of a double Beam girder to Flanges of Z Bars.

20" I Beams
88 Tons.

15" and 12"
I Beams
53 Tons.

10½", 10," 9" and 8"
I Beams
35 Tons.

7" and 6"
I Beams
17.6 Tons.

The number of tons indicated, denote the loads on single beams or girders for which the connections are proportioned.

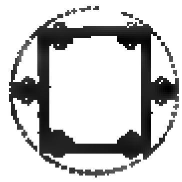
Rivets and Bolts ¾ dia.---All Bolts have beveled heads.

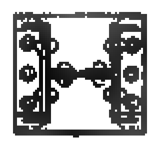
DETAILS OF STANDARD CONNECTIONS OF I BEAMS TO Z-BAR COLUMNS.

Connection of a single I beam to Webs of Z Bars.
Fig. 1

Splice of Z-Bar Column.
Fig. 3

Special Type of Z-Bar Column.
Fig. 2

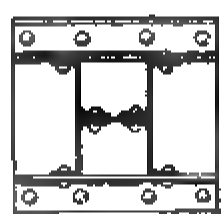




nnection of a double I beam girder to Webs of Z Bars.
Fig. 4

Base of Z-Bar Columns.
Fig. 5

Fig. 6



Number of rivets required for connections of different sizes of beams to webs of Z bars, will be the same as shown on preceding page, for similar beams to flanges of Z bars.

closed or box column. A form of column, offering advantages in some cases, especially if the column is to be finished circular in form, is shown by Fig. 2 on page 281. Fig. 3 on the same page shows the manner of splicing columns, whether of equal or unequal size.

"The standard connections for double I-beam girders and single floor beams to Z-bar columns, detailed on pages 280 and 281, were designed to fairly cover the range of ordinary practice. When the maximum loads in tons indicated for each case are exceeded, the connections may be correspondingly strengthened by simply using longer vertical angles for the brackets and increasing the number of rivets. In proportioning these connections, the shearing strain on rivets was assumed of a maximum intensity of 10,000 pounds per square inch. For steel Z-bar columns, the maximum loads given for these connections may be safely increased 15 per cent." *

The following tables give the safe load in tons for standard Z-bar columns of different lengths, as manufactured by Carnegie, Phipps & Co.

The values for steel Z-bar columns should be used only for cases in which the loads are for the most part statical, and equal, or very nearly so, on opposite sides of the columns. When there is much eccentricity of loading, or the loads are subject to sudden changes, the tabulated values must be reduced according to circumstances.

The Carnegie Steel Co. has discontinued the manufacture of iron bars of all kinds, and their product is now confined entirely to steel, which has practically superseded iron in structural work, being sold at the same price per pound, while 20 per cent. stronger.

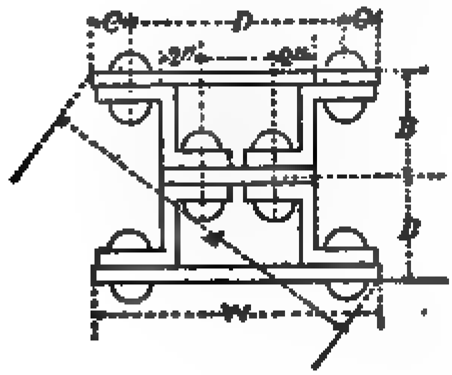
(The steel here referred to is what is known as "mild" steel, having an ultimate strength of about 60,000 pounds per square inch, and containing a comparatively low percentage of carbon.)

Example.—What size of Z-bar column, 30 feet long, with square bearing ends, will be required to carry a load of 200 tons, using a safety factor of 4?

Ans. Referring to table of steel Z-bar columns, page 287, we find that for a length of 30 feet, a 12-inch column with $\frac{5}{8}$-inch metal, weighing 118.4 lbs. per foot, will support with safety 202.6 tons, which is slightly in excess of the load.

* Carnegie, Phipps & Co.'s Pocket Companion, 1890.

Z-BAR COLUMN DIMENSIONS.



Dimensions of Z-Bar columns in inches for minimum and maximum thicknesses.

NOTE.—In columns A, B, C, D, E, and F, the thickness of the Z-bars and web plates does not vary, the variations in the strength of the column being made in the thickness of the side plates.

Columns G, H, K, and L, have no side plates, and the variations are in the thickness of the bars and web plate.

All of Column B and part of A have four side plates, two on each side, the others have but one plate on each side.

SAFE LOADS IN TONS OF 2,000 LBS.

STEEL Z-BAR COLUMNS.

Square Ends.

Allowed strains per square inch ; safety factor 4 : $\begin{cases} \text{12,000 lbs., for length of 90 radii or under.} \\ 17{,}100-57\dfrac{l}{r}\text{, for lengths over 90 radii.} \end{cases}$

20″ Z-BAR COLUMNS.—A.

Section : 4 Z-Bars $6\frac{1}{4}''$ × $\frac{3}{4}''$. 1 Web Plate 14″ × 1″. Side Plates 20″ wide.

20″ Z-BAR COLUMNS.—B.

Section : 4 Z-Bars $6\frac{1}{4}''$ × $\frac{7}{8}''$ 1 Web Plate 14″ × 1″. 4 Side Plates 20″ wide.

SAFE LOADS IN TONS OF 2,000 LBS.

STEEL Z-BAR COLUMNS.

Square Ends.

Allowed strains per square inch; safety factor 4: { 12,000 lbs., for lengths of 90 radii or under. $17,100-57\frac{l}{r}$, for lengths over 90 radii

16″ Z-BAR COLUMNS.—D.

Section: 4 Z-Bars 6¼″ × ¾″. 1 Web Plate 10″ × 1″. 2 Side Plates 16″ wide.

18″ Z-BAR COLUMNS.—C.

Section: 4 Z-Bars 6¼″ × ¾″. 1 Web Plate 12″ × 1″. 2 Side Plates 18″ wide.

SAFE LOADS IN TONS OF 2,000 LBS.

STEEL Z-BAR COLUMNS.

Square Ends.

Allowed strains per square inch; safety factor 4: { 12,000 lbs., for lengths of 90 radii or under. 17,100-57 $\frac{l}{r}$, for lengths over 90 radii.

14″ Z-BAR COLUMNS.—E.

Section: 4 Z-Bars 6⅛″ × 11/16″. 1 Web Plate 8″ × 11/16″. 2 Side Plates 14″ wide.

14″ Z-BAR COLUMNS.—F.

Section: 4 Z-Bars 6″ × ¾″. 1 Web Plate 8″ × ¾″. 2 Side Plates 14″ wide.

40 .....

42 ....

44 ....

213

20

SAFE LOADS IN TONS OF 2,000 LBS.

STEEL Z-BAR COLUMNS.

Square Ends.

Allowed strains per square inch for steel; safety factor 4:	12,000 lbs., for lengths of 90 radii or under. $17{,}100-57\frac{l}{r}$, for lengths over 90 radii.

12″ STEEL Z-BAR COLUMNS.—G.

Section: 4 Z-bars 6″ deep and 1 Web Plate 8″ x thickness of Z-bars.

26 and under	123.2	150 4	173 6	187.3	209 0	231.0	243.1	264.4	286.1
28	126.9	149.8	172.6	186.1	279 0	231 0	242.1	264 4	286 1
30	122.9	145.1	167 6	180.4	202 6	225.2	234.7	257.1	279 9
32	118.9	140.6	162.4	174.6	196 2	218 3	227.2	249.1	271.4
34	114.9	135.9	157 2	168.8	189.9	211.4	219 8	241 2	262 9
36	111.0	131.3	152.0	163.1	183.5	204.4	212.3	233 2	254 4
38	107.0	126.7	146.7	157 3	177.2	197 5	204 9	225 2	245 9
40	103.0	122.1	141 5	151.5	170.8	190.6	197.4	217.3	237.3
42	99.0	117.5	136.3	145.7	164.4	183 7	190 0	209.3	228.8
44	95.0	112 9	131.1	140 0	158.1	176 7	182 5	201.3	220.3
46	91.1	108.3	125.9	134.2	151.7	169 8	175 1	193.4	211 8
48	87 1	103 7	120 7	128.4	145.4	162.9	167.6	185 4	203.3
50	83.1	99.1	115.4	122.6	139.0	155.9	160.2	177.4	194.8

10″ STEEL Z-BAR COLUMNS.—H.

Section: 4 Z-bars 5″ deep and 1 Web Plate 7″ x thickness of Z-bars.

SAFE LOADS IN TONS OF 2,000 LBS.

STEEL Z-BAR COLUMNS.

Square Ends.

Allowed strains per square inch for steel; safety factor 4 { 12,000 lbs., for lengths of 90 radii or under. 17,100-57 $\frac{l}{r}$, for lengths over 90 radii.

8" STEEL Z-BAR COLUMNS.—K.

Section: 4 Z-bars 4" deep and 1 Web Plate 6" × thickness of Z-bars.

6" STEEL Z-BAR COLUMNS.—L.

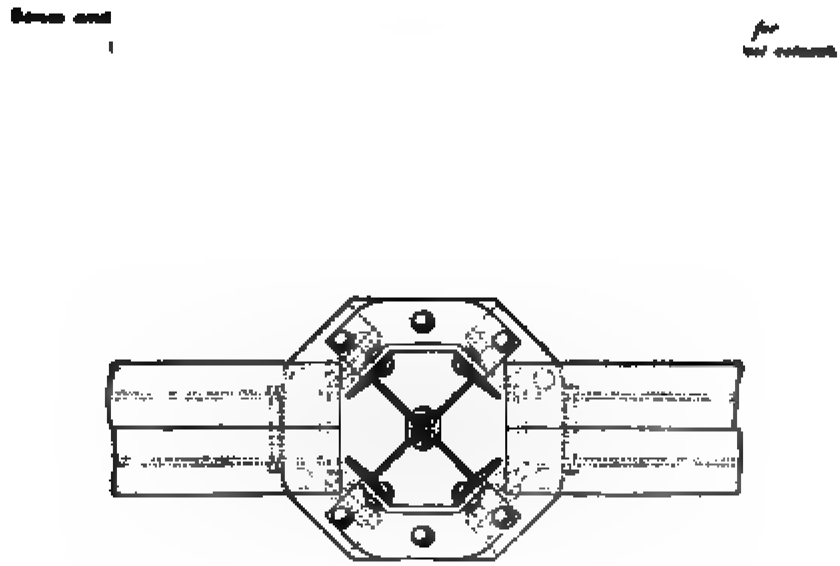
Section: 4 Z-bars 3" deep and 1 Web Plate 5¼" × thickness of Z-bars.

Length of column in feet	¼" metal 30.8 lbs.=9.1 sq. in. r (min.)=1.87.	5/16" metal 38.6 lbs.=11.4 sq. in. r (min.)=1.92.	⅜" metal 45.3 lbs.=13.3 sq. in. r (min.)=1.90.	7/16" metal 53.2 lbs.=15.7 sq. in. r (min.)=1.94.	½" metal 59.2 lbs.=17.4 sq. in. r (min.)=1.90.
12 and under	54.4	68.4	80.0	94.0	104.8
14	54.3	68.4	80.0	94.0	104.3
16	51.0	65.0	75.4	89.7	98.9
18	47.7	60.9	70.6	84.2	93.6
20	44.4	56.9	65.7	78.7	86.4
22	41.1	52.8	60.9	73.2	80.3
24	37.7	48.7	56.1	67.6	74.0
26	34.4	44.7	51.3	62.1	67.7
28	31.1	40.6	46.4	56.6	[illegible]
30	27.8	36.6	41.6	51.1	[illegible]

LARIMER'S PATENT ALL STEEL COLUMN.

(Manufactured by Jones & Laughlins, Pittsburgh.)

This column was patented June 2, 1891. It is made by bending two I-beams at right angles in the middle of the web and riveting



LARIMER'S PATENT ALL STEEL COLUMN.

them together as in the illustration. The column is very light and compact, and has but one row of rivets. The following table gives the strength of the column.

SAFE LOAD IN TONS OF 2,000 POUNDS, LARIMER'S PATENT ALL STEEL COLUMN.

No. 1. The allowed working strains being one-fourth of the ultimate strength of the column.

				Length of Column.															
				26′		24′		22′		20′		18′		16′		14′		12′	
Size of Column	Weight per Lineal Foot, Pounds	Size of I Beam	Weight of I Beam per Foot	Allowed Strains per □″ in Lbs.	Safe Load in Tons.	Allowed Strains per □″ in Lbs.	Safe Load in Tons.	Allowed Strains per □″ in Lbs.	Safe Load in Tons.	Allowed Strains per □″ in Lbs.	Safe Load in Tons.	Allowed Strains per □″ in Lbs.	Safe Load in Tons.	Allowed Strains per □″ in Lbs.	Safe Load in Tons.	Allowed Strains per □″ in Lbs.	Safe Load in Tons.	Allowed Strains per □″ in Lbs.	Safe Load in Tons.
16″	137.3	15″	61.0	12,000	229.0	12,000	229.0	12,000	229.0	12,000	229.0	12,000	229.0	12,000	229.0	12,000	229.0	12,000	229.0
16″	107.3	15″	48.0	12,000	186.0	12,000	186.0	12,000	186.0	12,000	186.0	12,000	186.0	12,000	186.0	12,000	186.0	12,000	186.0
16″	114.3	15″	5[illegible].5	12,000	217.0	12,000	217.0	12,000	217.0	12,000	217.0	12,000	217.0	12,000	217.0	12,000	217.0	12,000	217.0
16″	89.3	15″	39.0	12,000	153.0	12,000	153.0	12,000	153.0	12,000	153.0	12,000	153.0	12,000	153.0	12,000	153.0	12,000	153.0
16″	100.3	15″	47.5	12,000	184.0	12,000	184.0	12,000	184.0	12,000	184.0	12,000	184.0	12,000	184.0	12,000	184.0	12,000	184.0
13″	84.8	12″	38.0	12,000	150.0	12,000	150.0	12,000	150.0	12,000	150.0	12,000	150.0	12,000	150.0	12,000	170.0	12,000	150.0
13″	104.8	12″	48.0	12,000	186.0	12,000	186.0	12,000	186.0	12,000	186.0	12,000	186.0	12,000	186.0	12,000	186.0	12,000	186.0
13″	68.8	12″	31.0	12,000	121.0	12,000	121.0	12,000	121.0	12,000	121.0	12,000	121.0	12,000	121.0	12,000	121.0	12,000	121.0
13″	83.8	12″	37.5	12,000	149.0	12,000	149.0	12,000	149.0	12,000	149.0	12,000	149.0	12,000	149.0	12,000	149.0	12,000	149.0
11″	72.8	10″	32.0	11,[illegible]	123.0	12,000	128.0	12,000	128.0	12,000	128.0	12,000	128.0	12,000	128.0	12,000	128.0	12,000	128.0
11″	88.8	10″	40.0	11,772	147.0	11,628	173.0	12,000	157.0	12,000	157.0	12,000	157.0	12,000	157.0	12,000	157.0	12,000	157.0
11″	56.4	10″	23.8	11,296	9[illegible].5	11,8[illegible]5	97.0	12,000	99.0	12,000	99.0	12,000	99.0	12,000	99.0	12,000	99.0	12,000	99.0
11″	71.8	10″	31.5	10,950	116.0	11,117	121.0	11,913	126.0	12,000	127.0	12,000	127.0	12,000	127.0	12,000	127.0	12,000	127.0
10″	57.8	9″	2[illegible]	11,132	94.0	11,588	98.0	12,000	101.0	12,000	101.0	12,000	101.0	12,000	101.0	12,000	101.0	12,000	101.0
10″	68.8	9″	30.0	10,836	110.0	11,7[illegible]0	111.0	11,8[illegible]0	119.0	12,000	121.0	12,000	121.0	12,000	121.0	12,000	121.0	12,000	121.0
10″	48.3	9″	19.75	11,007	78.0	11,480	81.5	12,000	85.0	12,000	85.0	12,000	85.0	12,000	85.0	12,000	85.0	12,000	85.0
10″	57.8	9″	24.5	10,836	91.5	11,320	95.5	11,8[illegible]0	99.5	12,000	101.0	12,000	101.0	12,000	101.0	12,000	101.0	12,000	101.0

The Gray Column.

The figures on the opposite page show a perspective view and section of a column which was patented in December, 1892, by Mr. J. H. Gray, C. E., and which has since been used in some prominent buildings. As may be seen from the illustrations, this column is made of angle-bars riveted together and braced every few feet in height by flat iron ties, as shown in the perspective view.

The angles may be reënforced by cover-plates riveted to their faces, when necessary to increase the strength of the column. Any bridge-shop can make these columns by paying a small royalty to the patentee.

As angles are the cheapest shape of rolled steel that is manufactured, this should be an economical column.

The special advantages claimed for this column are:

1. A strong, economical section.
2. Provides continuous pipe space from basement to roof.
3. Has four flat sides for connections.
4. Size of column does not vary when section is increased or diminished.
5. Does away with "cap-plates," and joins sections of columns firmly together, making a continuous column.

Tests made in the hydraulic machine of the Keystone Bridge Works on 14-inch columns, 11 feet long, developed a resistance to crushing of from 38,000 to 40,000 pounds per square inch of section, and a modulus of elasticity of from 24,030,000 to 27,750,000 pounds.

The tables on pages 289*e*–289*h* give the safe loads of several sizes of square, wall, and corner columns as computed by Mr. Gray.

By varying the thickness of angles and adding cover-plates, the strength of the column can be greatly increased.

Tables of wall and corner columns, and further particulars, may be obtained by addressing Mr. J. H. Gray, C. E., Chicago.

SQUARE SECTION.

WALL SECTION.

CORNER SECTION.

SAFE LOADS IN TONS OF 2,000 LBS. BY FORMULA 17,100 LBS. $- 57 \frac{l}{r}$.

SQUARE COLUMNS WITHOUT COVER PLATES.

10″ COLUMN.

No. Pieces.	Dimensions.	Thick.	Area, Sq. in.	r.	12 ft.	16 ft.	20 ft.	30 ft.
8	$2\frac{1}{2}'' \times 2\frac{1}{2}''$ Ls.	$\frac{1}{4}$	9.52	3.16	69.0	64.1	60.8	55.3
8	" "	$\frac{5}{16}$	11.76	3.15	85.2	80.1	75.0	62.3
8	" "	$\frac{3}{8}$	13.84	3.13	100.0	94.2	88.1	73.0
8	" "	$\frac{7}{16}$	16.00	3.12	116.0	108.8	101.7	84.2
8	" "	$\frac{1}{2}$	18.00	3.11	130.1	122.2	114.3	94.5
8	$2\frac{1}{2}'' \times 3''$ Ls.	$\frac{1}{2}$	20.00	3.00	143.4	134.5	125.4	102.6
8	" "	$\frac{9}{16}$	22.24	2.98	159.5	149.3	139.1	113.6

12″ COLUMN.

No. Pieces.	Dimensions.	Thick.	Area, Sq. in.	r.	12 ft.	16 ft.	20 ft.	30 ft.
8	$3'' \times 3''$ Ls.	$\frac{1}{4}$	11.52	3.81	86.1	81.9	77.8	67.5
8	" "	$\frac{5}{16}$	14.24	3.79	106.3	101.2	96.1	83.2
8	" "	$\frac{3}{8}$	16.88	3.77	125.9	119.9	113.7	98.4
8	$3'' \times 4''$ Ls.	$\frac{3}{8}$	19.84	3.57	149.2	141.5	133.7	114.4
8	" "	$\frac{7}{16}$	22.96	3.55	169.8	160.9	152.1	129.9
8	$3'' \times 5''$ Ls.	$\frac{7}{16}$	26.48	3.36	194.1	183.2	172.5	145.5
8	" "	$\frac{1}{2}$	30.00	3.34	219.6	207.3	195.1	164.3
8	" "	$\frac{9}{16}$	33.44	3.32	244.6	230.8	217.5	183.2
8	" "	$\frac{5}{8}$	36.88	3.30	269.5	254.2	238.9	200.7
8	" "	$\frac{11}{16}$	40.24	3.28	293.7	276.9	260.1	218.2
8	" "	$\frac{3}{4}$	43.52	3.26	317.3	299.0	280.7	235.0
8	" "	$\frac{13}{16}$	46.72	3.24	340.3	320.6	300.8	251.5

14″ COLUMN.

No. Pieces.	Dimensions.	Thick.	Area, Sq. in.	r.	12 ft.	16 ft.	20 ft.	30 ft.
8	$4'' \times 3''$ Ls.	$\frac{5}{16}$	16.72	4.63	128.2	123.5	118.4	105.8
8	" "	$\frac{3}{8}$	19.84	4.61	152.0	146.1	140.2	125.4
8	$4'' \times 3\frac{1}{2}''$ Ls.	$\frac{3}{8}$	21.36	4.50	163.1	156.9	150.2	133.9
8	$4'' \times 4''$ Ls.	$\frac{3}{8}$	22.88	4.40	174.3	167.2	160.1	142.2
8	" "	$\frac{7}{16}$	26.48	4.39	201.7	193.4	185.2	164.5
8	$4'' \times 5''$ Ls.	$\frac{7}{16}$	30.00	4.12	226.6	216.7	206.7	181.8
8	" "	$\frac{1}{2}$	34.00	4.10	256.7	245.3	234.0	205.6
8	$4'' \times 6''$ Ls.	$\frac{1}{2}$	38.00	3.93	285.2	272.0	258.7	225.5
8	" "	$\frac{9}{16}$	42.48	3.92	321.7	306.7	291.8	254.2
8	" "	$\frac{5}{8}$	46.88	3.91	351.6	335.2	318.8	277.6
8	" "	$\frac{11}{16}$	51.29	3.89	384.4	366.3	348.4	303.3
8	" "	$\frac{3}{4}$	55.52	3.88	416.0	396.3	376.8	327.9
8	" "	$\frac{13}{16}$	59.76	3.87	447.6	426.4	405.3	352.5
8	" "	$\frac{7}{8}$	63.92	3.86	478.5	455.9	433.2	376.6

SAFE LOADS IN TONS OF 2,000 LBS. BY FORMULA 17,100 LBS. $- 57 \frac{l}{r}$.

SQUARE COLUMNS WITHOUT COVER PLATES.

16″ COLUMN.

No. Pieces.	Dimensions.	Thick.	Area Sq. In.	*r*.	12 ft.	16 ft.	20 ft.	30 ft.
8	5″ × 3″ Ls.	3/8	22.88	5.45	178.4	172.7	166.9	152.5
8	5″ × 3½″ Ls.	3/8	24.40	5.35	190.8	184.6	178.3	162.6
8	5″ × 4″ Ls.	3/8	25.84	5.24	200.7	194.0	187.2	170.4
8	" "	7/16	30.00	5.21	232.8	225.0	217.1	197.4
8	5″ × 5″ Ls.	7/16	33.44	5.01	258.5	249.4	240.2	217.5
8	" "	1/2	38.00	5.00	293.7	283.4	272.8	246.9
8	" "	9/16	42.44	4.98	328.2	316.5	304.9	275.7
8	" "	5/8	46.88	4.96	362.1	349.2	336.2	303.9
8	" "	11/16	51.36	4.94	396.4	382.3	368.0	332.5
8	" "	3/4	55.52	4.93	428.5	413.1	397.7	359.2
8	" "	13/16	59.68	4.92	460.5	443.9	427.3	385.9

18″ COLUMN.

No. Pieces.	Dimensions.	Thick.	Area Sq. In.	*r*.	12 ft.	16 ft.	20 ft.	30 ft.
8	6″ × 3½″ Ls.	3/8	27.36	6.15	215.7	209.6	203.5	188.3
8	6″ × 4″ Ls.	3/8	28.88	6.07	227.4	220.9	214.4	198.1
8	" "	7/16	33.44	6.05	263.2	255.7	248.1	229.3
8	" "	1/2	38.00	6.03	299.0	290.4	281.8	260.2
8	6″ × 6″ Ls.	7/16	40.48	5.64	316.6	306.8	297.0	272.5
8	" "	1/2	46.00	5.63	359.8	348.6	337.4	309.5
8	" "	9/16	51.44	5.62	402.5	389.7	377.2	345.9
8	" "	5/8	56.88	5.60	444.6	430.7	416.8	382.1
8	" "	11/16	62.24	5.59	486.5	471.3	456.1	417.9
8	" "	3/4	67.52	5.57	527.3	511.0	494.4	452.9
8	" "	13/16	72.72	5.55	568.0	550.0	532.0	487.3
8	" "	7/8	77.92	5.54	608.5	589.2	569.9	521.9

22″ COLUMN.

No. Pieces.	Dimensions.	Thick.	Area Sq. In.	*r*.	12 ft.	16 ft.	20 ft.	30 ft.
8	8″ × 6″ Ls.	1/2	54.00	7.30	431.4	421.3	411.1	385.8
8	" "	9/16	60.48	7.29	483.1	471.8	460.4	432.0
8	" "	5/8	66.88	7.27	534.2	521.5	508.9	477.5
8	" "	11/16	73.28	7.26	585.2	571.3	557.5	523.0
8	" "	3/4	79.52	7.24	634.8	619.8	604.8	567.3
8	" "	13/16	85.76	7.23	684.6	668.4	652.1	611.6
8	" "	7/8	91.92	7.22	733.6	716.3	698.8	655.3
8	" "	15/16	98.08	7.21	782.7	764.2	745.6	699.0
8	" "	1	104.16	7.20	831.2	812.4	791.6	742.2

SAFE LOADS IN TONS OF 2,000 LBS. BY FORMULA 17,100 LBS. $- 57\frac{l}{r}$.

WALL COLUMNS WITHOUT COVER PLATES.

10″ COLUMN.

No. Pieces.	Dimensions.	Thick.	Area sq. in.	r.	12 ft.	16 ft.	20 ft.	30 ft.
6	2½″ × 2½″ Ls.	1/4	7.14	2.25	48.0	43.7	39.3	28.5
6	" "	5/16	8.82	2.25	59.3	53.9	48.6	35.2
6	" "	3/8	10.38	2.24	69.7	63.4	57.1	41.2
6	" "	7/16	12.00	2.24	80.6	73.3	65.0	47.6
6	" "	1/2	13.50	2.23	90.6	81.9	74.0	53.3
6	2½″ × 3″ Ls.	1/2	15.00	2.17	99.9	90.4	81.0	57.0
6	" "	9/16	16.68	2.16	110.9	100.3	89.8	63.3

12″ COLUMN.

No. Pieces.	Dimensions.	Thick.	Area sq. in.	r.	12 ft.	16 ft.	20 ft.	30 ft.
6	3″ × 3″ Ls.	1/4	8.64	2.71	60.8	56.4	52.0	41.2
6	" "	5/16	10.68	2.70	75.1	69.7	64.3	50.7
6	" "	3/8	12.66	2.69	88.9	82.5	76.1	59.9
6	3″ × 4″ Ls.	3/8	14.88	2.56	103.4	95.4	87.4	67.6
6	" "	7/16	17.22	2.55	119.7	110.4	101.1	78.0
6	3″ × 5″ Ls.	7/16	19.86	2.47	136.8	125.8	114.8	87.3
6	" "	1/2	22.50	2.47	155.0	142.5	130.0	98.9
6	" "	9/16	25.05	2.46	172 6	158.6	144.6	109.8
6	" "	5/8	27.66	2.46	190.3	174.9	159.5	121.1
6	" "	11/16	30.18	2.45	207.4	190.6	173.8	131.6
6	" "	3/4	32 64	2.44	224.1	205.8	187.5	141.8
6	" "	13/16	35.04	2.43	240.4	220.7	201.0	151.6

14″ COLUMN.

No. Pieces.	Dimensions.	Thick.	Area sq. in.	r.	12 ft.	16 ft.	20 ft.	30 ft.
6	4″ × 3″ Ls.	5/16	12.54	3.33	91.8	86.7	81.5	74.8
6	" "	3/8	14.88	3.31	108.8	102.7	96.6	88.5
6	4″ × 3½″ Ls.	3/8	16.02	3.25	116.7	110.0	103.3	86.4
6	4″ × 4″ Ls.	3/8	17.16	3.19	124.6	117.3	109.9	91.4
6	" "	7/16	19.86	3.18	144.2	135.6	127.1	105.5
6	4″ × 5″ Ls.	7/16	22.50	3.06	166.2	152.1	142.1	116.9
6	" "	1/2	25.50	3.05	183.7	172.3	160.9	132.2
6	4″ × 6″ Ls.	1/2	28.50	2.97	204.3	191.2	176.1	145.2
6	" "	9/16	31.86	2.96	228.2	213.5	198.7	161.9
6	" "	5/8	35.16	2.95	251.7	235.4	219.1	178.3
6	" "	11/16	38 47	2.95	275.3	257.6	239.7	195.1
6	" "	3/4	41.64	2.94	297.9	278.6	259.3	210.7
6	" "	13/16	44.82	2.94	320.7	299.7	278.7	226.8
6	" "	7/8	47.94	2.93	342.7	320.4	298.1	242.0

SAFE LOADS IN TONS OF 2,000 LBS. BY FORMULA $17{,}100 - 57\frac{l}{r}$.

CORNER COLUMNS WITHOUT COVER PLATES.

11″ COLUMN REDUCED FROM 14″ COLUMN.

15″ COLUMN REDUCED FROM 18″ COLUMN.

4	6″ × 3½″ L.	3/8	13.68	5″ × 5″	3/8	105.1	101.2	97.2	87.8
4	6″ × 4″ L.	3/8	14.44	" "	3/8	110.7	106.4	102.1	91.5
4	" "	7/16	16.72	" "	7/16	128.2	123.8	118.4	106.1
4	" "	1/2	19.00	" "	1/2	145.7	140.1	134.5	120.6
4	6″ × 6″ L.	7/16	20.24	" "	7/16	154.1	147.7	141.2	125.3
4	" "	1/2	23.00	" "	1/2	175.0	167.7	160.5	142.4
4	" "	9/16	25.72	" "	9/16	195.6	187.5	179.4	159.2
4	" "	5/8	28.44	" "	[illegible]	216.3	207.3	198.3	176.1
4	" "	11/16	31.12	" "	[illegible]	236.0	226.0	216.0	191.0
4	" "	3/4	33.76	" "	[illegible]	255.2	244.0	232.8	205.0
4	" "	13/16	36.36	" "	[illegible]	273.8	261.6	249.4	218.5
4	" "	7/8	38.96	" "	[illegible]	292.5	279.0	265.5	231.6

CHAPTER XII.

BENDING-MOMENTS.

THE bending-moment of a beam or truss represents the destructive energy of the load on the beam or truss at any point for which the bending-moment is computed.

The moment of a force around any given axis is the product of the force into the perpendicular distance between the line of action of the force and the axis, or the product of the force into its arm.

In a beam the forces or loads are all vertical and the arms horizontal.

The bending-moment at any cross-section of a beam is the algebraic sum of the moments of the forces tending to turn the beam around the horizontal axis passing through the centre of gravity of the section.

EXAMPLE. — Suppose we have a beam with one end securely fixed into a wall, and the other end projecting from it, as in Fig. 1.

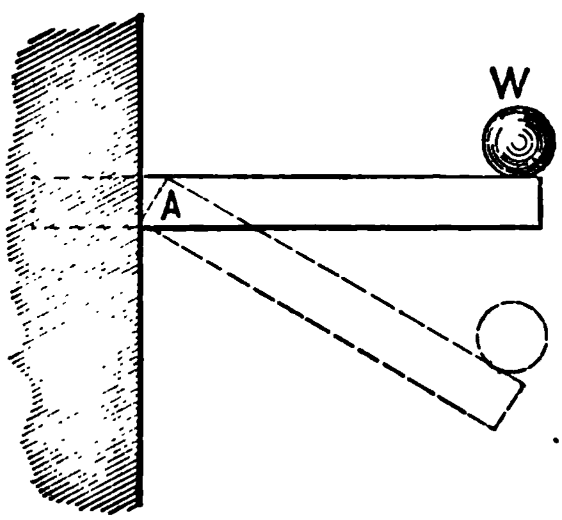


Fig. 1.

Let us now suppose we have a weight, which, if placed at the end of the beam, will cause it to break at the point of support.

Then, if we were to place the weight on the beam at a point near the wall, the beam would support the weight easily; but, as we move the weight towards the outer end of the beam, the beam bends more and more; and, when the weight is at the end, the beam breaks, as shown by the dotted lines, Fig. 1.

Now, it is evident that the destructive energy of the weight is greater, the farther the weight is removed from the wall-end of the beam, though the weight itself remains the same all the time. The reason for this is, that the moment of the weight tends to turn the beam about the point A, and thus produces a pull on the upper fibres of the beam, and compresses the lower fibres. As the weight is moved out on the beam, its moment becomes greater, and hence also the pull and compression on the fibres; and, when the

moment of the weight produces a greater tension or compression on the fibres than they are capable of resisting, they fail, and the beam breaks. Before the fibres break, however, they commence to stretch, and this allows the beam to bend: hence the name "bending-moment" has been given to the moment which causes a beam to bend, and perhaps ultimately to break.

There may, of course, be several loads on a beam, and each one having a different moment, tending to bend the beam; and it may also occur that some of the weights may tend to turn the beam in different directions: the algebraic sum of their moments (calling those tending to turn the beam to the right +, and the others —) would be the bending-moment of the beam.

Knowing the bending-moment of a beam, we have only to find the section of the beam that is capable of resisting it, as is shown in the general theory of beams, Chap. XIV.

To determine the bending-moments of beams mathematically, requires considerable training in mechanics and mathematics; but, as most beams may be placed under some one of the following cases, we shall give the bending-moment for these cases, and then show how the bending-moment for any other methods of loading may be easily obtained by a scale diagram.

Examples of Bending-Moments.

Case I.

Beam fixed at one end, and loaded with concentrated load W.

Bending-moment $= W \times L$. (L may, or may not, be the whole length of the beam, according to where the weight is located.)

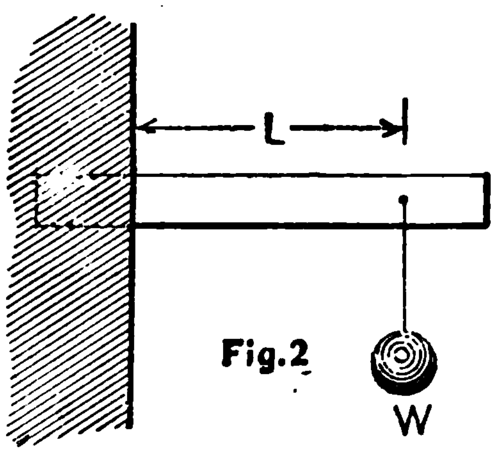


Fig.2

Case II.

Beam fixed at one end, loaded with a distributed load W.

$$\text{Bending moment} = W \times \frac{L}{2}.$$

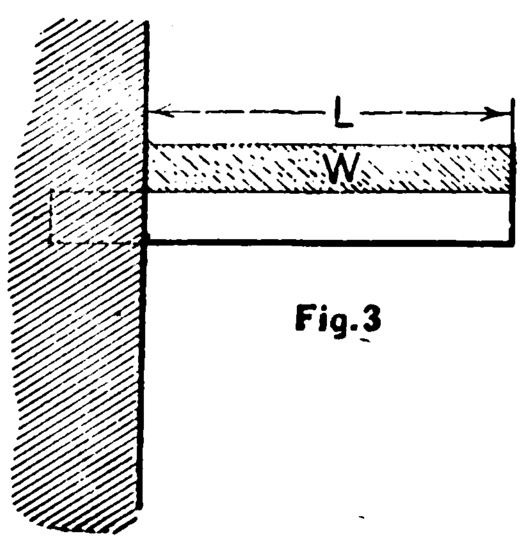


Fig.3

Note. — The length L must always be taken in the same unit of measurement as is used for the breadth and depth: thus, if B and D are in inches, L must be in inches.

Case III.

*Beam fixed at one end, loaded with **both a concentrated and a** distributed load.*

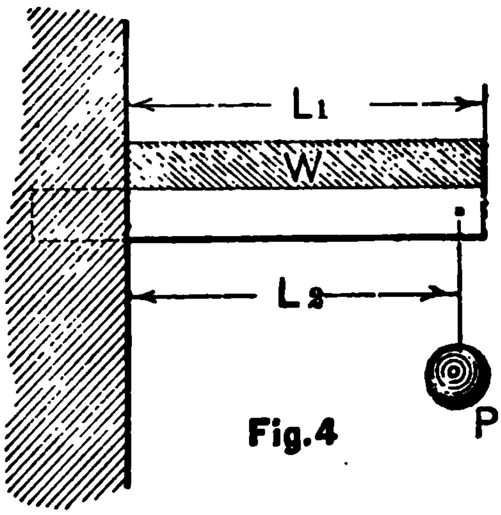


Fig. 4

Bending-moment $= P \times L_2 + W \times \frac{L_1}{2}$.

Case IV.

*Beam supported at both ends, loaded with **concentrated load** at centre.*

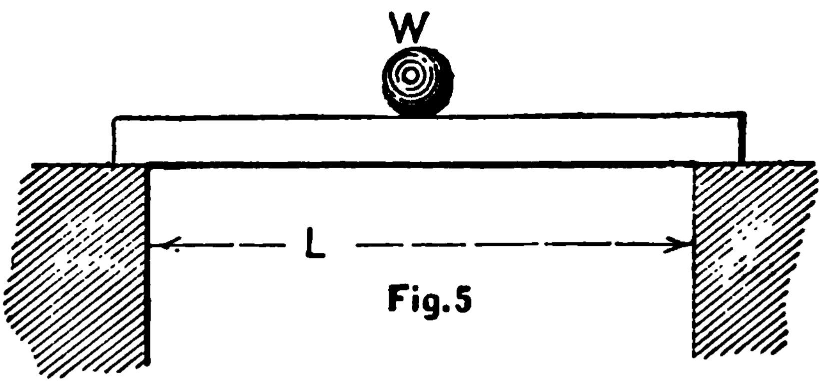


Fig. 5

Bending-moment

$= W \times \frac{L}{4}$.

Case V.

Beam supported at both ends, loaded with a distributed load W.

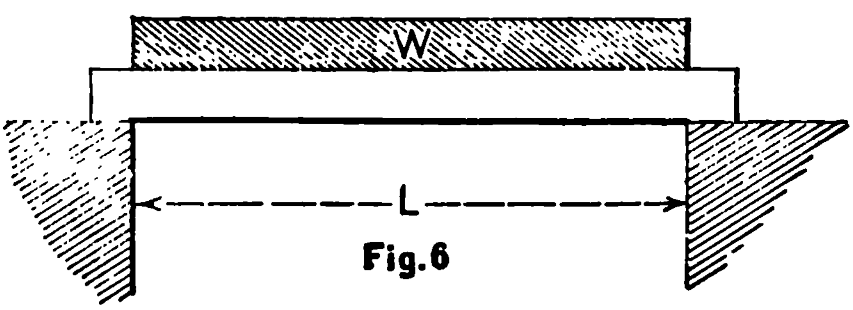


Fig. 6

Bending-moment

$= W \times \frac{L}{8}$.

Case VI.

*Beam supported at both ends, loaded with concentrated **load not** at centre.*

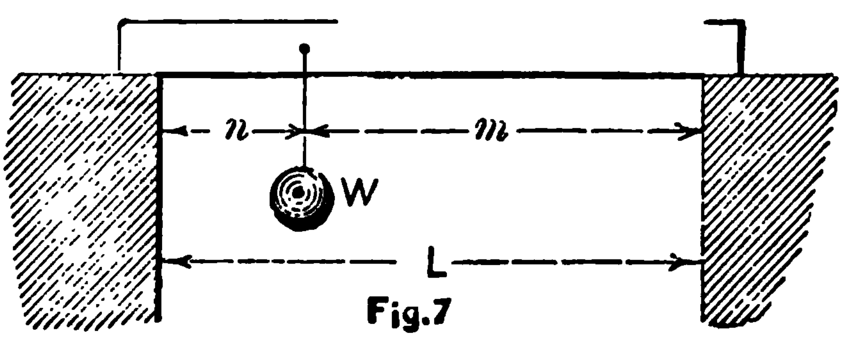


Fig. 7

Bending-moment

$= W \times \frac{m \times n}{L}$.

Case VII.

Beam supported at both ends, loaded with two equal concentrated loads, equally distant from the centre.

Bending-moment

$= W \times m.$

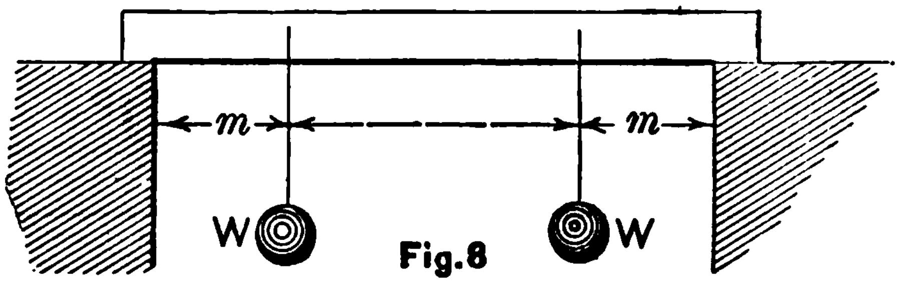


Fig. 8

From these examples it will be seen that all the quantities which enter into the bending-moment are the weight, the span, and the distance of point of application of concentrated load from each end.

The bending-moment for any case other than the above may easily be obtained by the graphic method, which will now be explained.

Graphic Method of Determining Bending-Moments.

The bending-moment of a beam supported at both ends, and loaded with one concentrated load, may be shown graphically, as follows: —

Let W be the weight applied, as shown. Then, by rule under Case VI., the bending-moment directly under

$W = W \times \dfrac{m \times n}{L}$.

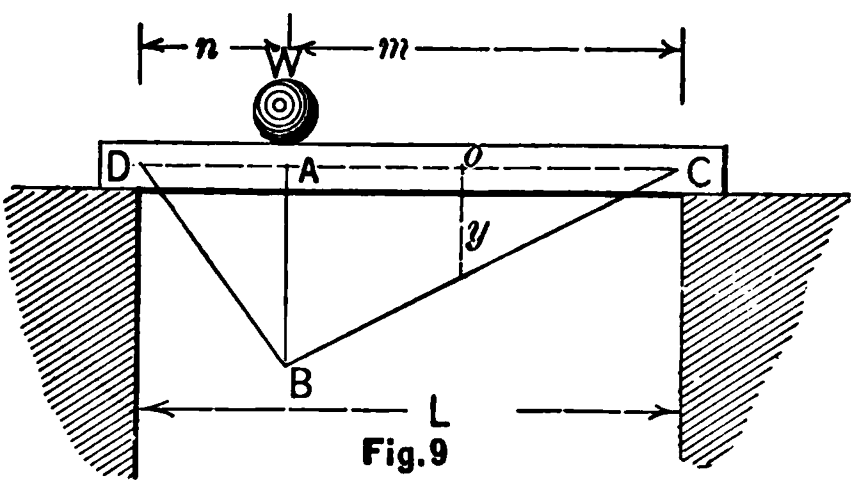


Fig. 9

Draw the beam, with the given span, accurately to scale, and then measure down the line AB equal to the bending-moment. Connect B with each end of the beam. If, then, we wished to find the bending-moment at any other point of the beam, as at o, draw the vertical line y to BC; and its length, measured to the same scale as AB, will give the bending-moment at o.

Beam with two concentrated loads.

To draw the bending-moment for a beam with two concentrated loads, first draw the dotted lines ABD and ACD, giving the outline

of the bending-moment for each load separately; EB being equal to $W \times \frac{m \times n}{L}$, and FC equal to $P \times \frac{r \times s}{L}$.

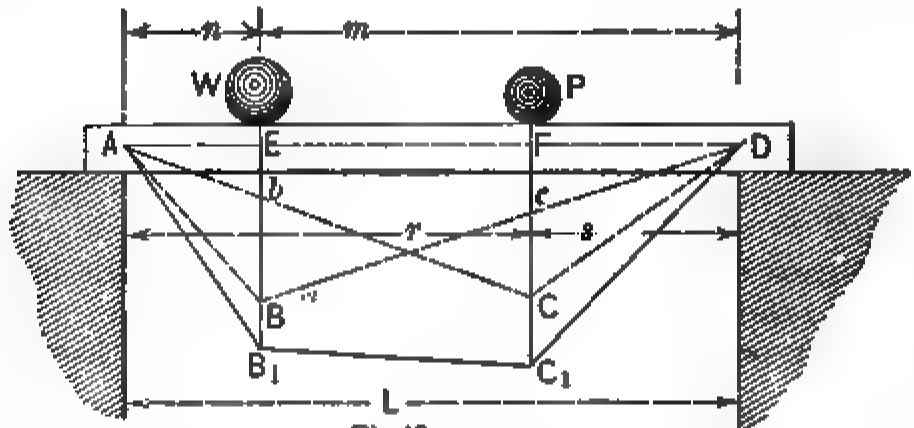


Fig. 10

Now, the bending-moment at the point E equals EB, due to the load W, and Eb, due to the load P: hence the bending-moment at E should be drawn equal to $EB + Eb = EB_1$; and at F the bending moment should equal $FC + Fc = FC_1$. The outline for the bending-moment due to both loads, then, would be the line AB_1C_1D, and the greatest bending-moment would in this particular case be FC_1.

Beam with three concentrated loads.

Fig. 11

Proceed as in the last case, and draw the bending-moment for each load separately. Then make $AD = A1 + A2 + A3$, $BE = B1 + B2 + B3$, and $CF = C1 + C2 + C3$. The line $HDEFI$ will then be the outline for the bending-moment due to all the weights. The bending-moment for a beam loaded with any number of concentrated weights may be drawn in the same way.

Beam with uniformly distributed load.

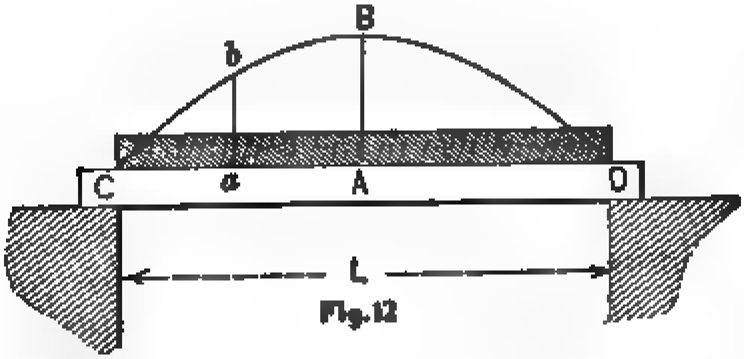


Fig. 12

Draw the beam with the given span, accurately to a scale, as before, and at the middle of the beam draw the vertical line AB equal to $W \times \frac{L}{8}$, W representing the whole distributed load. Then connect the points C, B, D by a parabola, and it will give the outline of the bending-moments. If, now, we wanted the bending-moment at the point a, we have only to draw the vertical line ab, and measure it to the same scale as AB, and it will be the moment desired. Methods for drawing the parabola may be found in "Geometrical Problems," Part I.

Beam loaded with both distributed and concentrated loads.

To determine the bending-moment in this case, we have only to combine the methods for concentrated loads and for the distributed load, as shown in the accompanying figure. The bending-moment at any point on the beam will then be limited by the line ABC on top, and $CDEFA$ on the bottom; and the greatest bending-moment will be the longest vertical line that can be drawn between these two bounding lines.

B

Fig. 13

For example, the bending-moment at X would be BE. The position of the greatest bending-moment will depend upon the position of the concentrated loads, and it may and may not occur at the centre.

EXAMPLE. — What is the greatest bending-moment in a beam of 20 feet span, loaded with a distributed load of 800 pounds and a concentrated load of 500 pounds 6 feet from one end, and a concentrated load of 600 pounds 7 feet from the other end?

Ans. 1st, The moment due to the distributed load is $W \times \frac{L}{8}$, or $\frac{800 \times 20}{8} =$ 2000 pounds. We therefore lay off to a scale, say 4000 pounds to the inch, $B1 =$ 2000 pounds, and draw a parabola between the points A, B, and C.

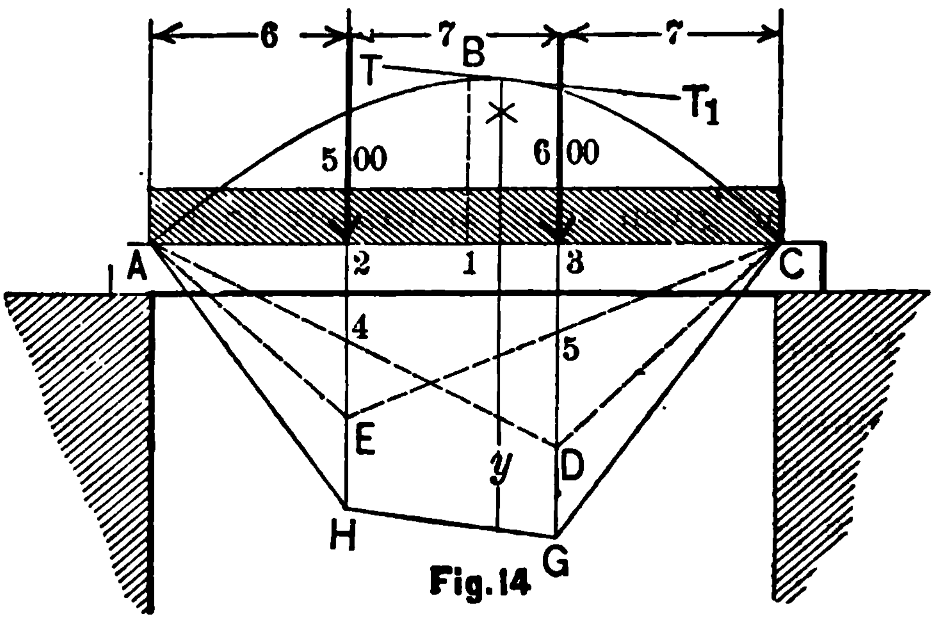


Fig. 14

2d, The bending-moment for the concentrated load of 500 pounds is $\frac{500 \times 6 \times 14}{20}$, or 2100 pounds. Hence we draw $E2 = 2100$ pounds, to the same scale as $B1$, and then draw the lines AE and CE.

3d, The bending-moment for the concentrated load of 600 pounds is $\frac{600 \times 7 \times 13}{20}$, or 2730 pounds; and we draw $D3 = 2730$ pounds, and connect D with A and C.

4th, Make $EH = 2 - 4$, and $DG = 3 - 5$, and connect G and H with C and A and with each other.

The greatest bending-moment will be represented by the longest vertical line which can be drawn between the parabola ABC and the broken line $AHGC$. In this example we find the longest vertical line which can be drawn is xy; and by scaling it we find the greatest bending-moment to be 5550 pounds, applied 10 feet 11 inches from the point A.

In this case, the position of the line Xy was determined by drawing the line TT_1 parallel to HG, and tangent to ABC. The line Xy is drawn through the point of tangency.

NOTE. — As the measurements used for determining the bending-moment are in feet, we must multiply the moment by 12, to get it into inch pounds; otherwise, in working out the dimensions of the beam, they would be in feet instead of inches.

CHAPTER XIII.

MOMENTS OF INERTIA AND RESISTANCE, AND RADIUS OF GYRATION.

Moment of Inertia.

The strength of sections to resist strains, either as girders or as posts, depends not only on the area, but also on the form of the cross-section. The property of the section which represents the effect of the form upon the strength of a beam or post is its moment of inertia, usually denoted by I. The moment of inertia for any cross-section is the sum of the products obtained by multiplying the area of each particle in the cross-section by the square of its distance from the neutral axis.

Note.—The neutral axis of a beam is the line on which there is neither tension nor compression; and, for wooden or wrought-iron beams or posts, it may, for all practical purposes, be considered as passing through the centre of gravity of the cross-section.

For most forms of cross-section the moment of inertia is best found by the aid of the calculus; though it may be obtained by dividing the figure into squares or triangles, and multiplying their areas by the squares of the distance of their centres of gravity from the neutral axis.

Moment of Resistance.

The resistance of a beam to bending and cross-breaking at any given cross-section is the moment of the two equal and opposite forces, consisting of the thrust along the longitudinally compressed layers, and the tension along the longitudinally stretched layers.

This moment, called "the moment of resistance," is, for any given cross-section of a beam, equal to

$$\frac{\text{moment of inertia}}{\text{extreme distance from axis}}.$$

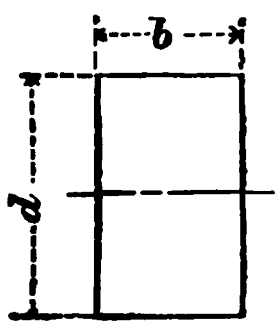
In the general formula for strength of columns, given on p. 231, the effect of the form of the column is expressed by the square of the **radius of gyration,** which is the moment of inertia of the section divided by its area; or $\frac{I}{A} = r^2$. The moments of inertia of the principal elementary sections, and a few common

forms, are given below, which will enable the moment about any given neutral axis for any other section to be readily calculated by merely adding together the moments about the given axis of the elementary sections of which it is composed.

In the case of hollow or re-entering sections, the moment of the hollow portion is to be subtracted from that of the enclosing area.

Moments of Inertia and Resistance, and Radii of Gyration.

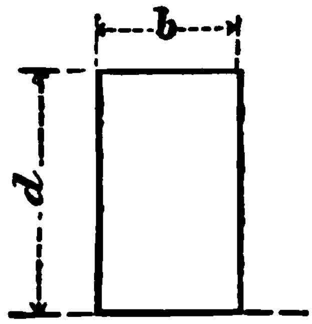
I = Moment of inertia.
R = Moment of resistance.
G = Radius of gyration.
A = Area of the section.
Position of neutral axis represented by broken line.



$$I = \frac{bd^3}{12}.$$

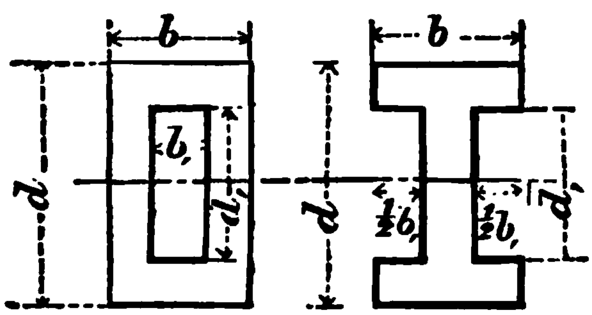
$$R = \frac{bd^2}{6}.$$

$$G^2 = \frac{d^2}{12}.$$



$$I = \frac{bd^3}{3}.$$

$$G^2 = \frac{d^2}{3}.$$



$$I = \frac{bd^3 - b_{,}d_{,}^3}{12}.$$

$$R = \frac{2I}{d}.$$

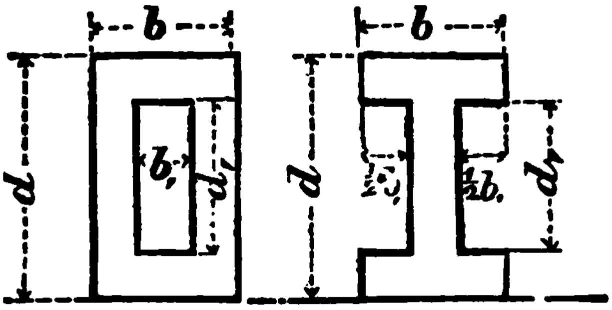
$$G^2 = \frac{I}{bd - b_{,}d_{,}}.$$

I-Beam (another formula).

Let a denote area of one flange,
a' denote area of web,
d' = effective depth between centres of gravity of flanges;

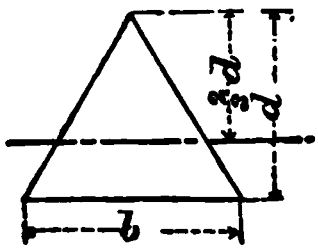
then
$$I = \left(a + \frac{a'}{6}\right)\frac{d'^2}{2}.$$

This is the formula generally used by the engineers for the iron companies.



$$I = \frac{bd^3}{3} - b_{,}d_{,}\frac{d^2}{4} - \frac{b_{,}d_{,}^3}{12}.$$

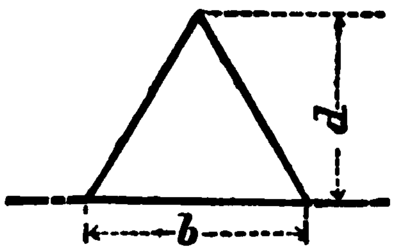
$$G^2 = \frac{I}{A}.$$



$$I = \frac{bd^3}{36}.$$

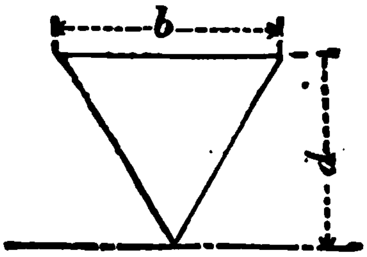
$$R = \frac{3I}{2d} = \frac{bd^2}{24}.$$

$$G^2 = \frac{I}{A} = \frac{d^2}{18}.$$



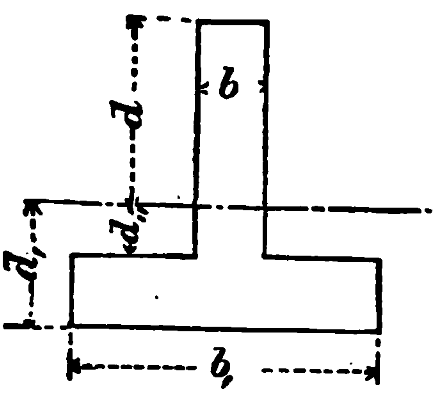
$$I = \frac{bd^3}{12}.$$

$$G^2 = \frac{d^2}{6}.$$



$$I = \frac{bd^3}{4}.$$

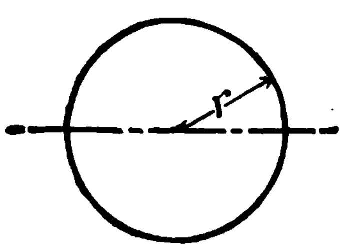
$$G^2 = \frac{d^2}{2}.$$



$$I = \frac{bd^3 + b_{,}d_{,}^3 - (b_{,} - b)d_{,,}^3}{3}.$$

$$R = \frac{I}{d}.$$

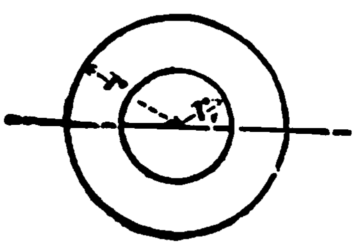
$$G^2 = \frac{I}{A}.$$



$$I = 0.7854r^4.$$

$$R = 0.7854r^3.$$

$$G^2 = \frac{r^2}{4}.$$



$$I = 0.7854\,(r^4 - r_{,}^4).$$

$$R = 0.7854\left(r^3 - \frac{r_{,}^4}{r}\right).$$

$$G^2 = \frac{1}{4}\,\frac{r^4 - r_{,}^4}{r^2 - r_{,}^2}.$$

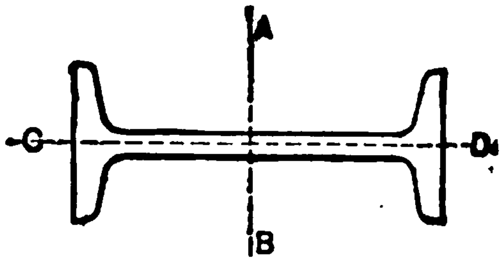
Moments of Inertia and Radii of Gyration of Merchant Shapes of Iron and Steel.

For the sections of rolled iron beams and bars to be found in the market, the moments of inertia are given in the "Book of Sections" published by the manufacturers. The following tables give the moments of inertia and radii of gyration for the principal sections manufactured by Carnegie, Phipps & Co., the New Jersey Steel and Iron Company, and the Phœnix Iron Company (revised to October 1, 1891). The Pencoyd Iron Works have recently made changes in a number of their sections, and some of the old sections of iron beams and channels have been abandoned, and they are not at present prepared to furnish the revised data.

The tables give the least weight for each section of iron beam, and the minimum and maximum weights for channels, deck beams, and angle irons. These shapes can be rolled for any weight between the two given, while the weight of the beams can also be greatly increased. With the quantities given in these tables, one can find all the data required in usual calculations.

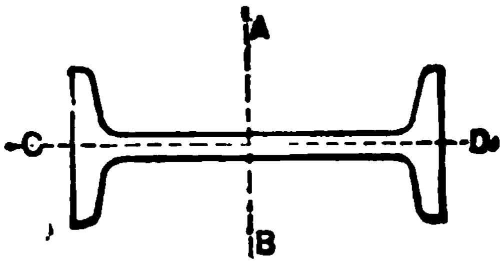
The tables on pages 322–24 will be found very convenient in computing the strength of struts formed of two or four angle bars.

MOMENTS OF INERTIA AND RADII OF GYRATION OF CARNEGIE BEAMS—IRON.



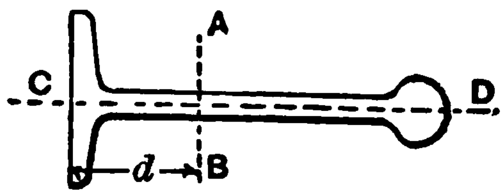
		I.	II.	III.	IV.	V.
Size, in inches.	Weight per foot, in lbs.	Area of cross-section, in sq. in.	Moments of inertia.		Radii of gyration.	
			Axis A B.	Axis C D.	Axis A B.	Axis C D.
15	80	24.0	813.7	38.8	5.82	1.27
15	60	18.0	625.5	23.0	5.90	1.13
15	50	15.0	522.6	15.5	5.90	1.02
12	56.5	17.0	348.5	17.4	4.53	1 01
12	42	12.6	274.8	11.0	4.67	0.94
10½	40	12.0	201.7	12.0	4.10	1.00
10½	31.5	9.5	165.0	8.01	4.17	0.92
10	42	12.6	198.8	13.74	3.97	1.04
10	36	10.8	170.6	10.02	3.97	0.96
10	30	9.0	145.8	7.43	4.03	0.91
9	38.5	11.6	150.1	12.84	3.61	1.05
9	28.5	8.6	110.3	6.79	3.59	0.89
9	23.5	7.1	92.3	4.64	3.62	0.81
8	34	10.2	102.0	10.2	3.16	0.99
8	27	8.1	82.5	6.30	3.19	0 88
8	21.5	6.5	66.2	3.95	3.20	0.78
7	22	6.6	51.9	4.58	2.80	0.83
7	18	5.4	44.2	3.28	2.86	0.78
6	16	4.8	29.0	2.87	2.46	0.77
6	13.5	4.1	24 4	2.00	2.46	0 70
5	12	3.6	14.4	1.46	2.00	0.64
5	10	3.0	12.5	1.15	2.04	0.62
4	7	2.1	5.7	0.67	1.65	0.57
4	6	1.8	4.6	0.36	1.61	0.45
3	9	2.7	3 5	0.85	1.15	0.56
3	5.5	1.7	2.5	0.44	1.24	0.52

MOMENTS OF INERTIA AND RADII OF GYRATION OF CARNEGIE BEAMS—STEEL.



Size, in inches.	Weight per foot, in lbs.	I. Area of cross-section, in sq. in.	II. Moments of inertia. Axis A B.	III. Moments of inertia. Axis C D.	IV. Radii of gyration. Axis A B.	V. Radii of gyration. Axis C D.
24	80	23.2	2,059.3	41.6	9.42	1.34
20	80	23.5	1,449.2	45.6	7.85	1.39
20	64	18.8	1,146.0	27.3	7.80	1.20
15	75	22.1	757 7	40 1	5.86	1.35
15	60	17.6	644.0	30.4	6.04	1.32
15	50	14.7	529.7	21.0	6.00	1.20
15	41	12.0	424.1	14.0	5.94	1.08
12	40	11.7	281.3	16.8	4.90	1.20
12	32	9.4	222.3	10.3	4.85	1.04
10	33	9.7	161.3	11.8	4.08	1.10
10	25.5	7.5	123.7	7.32	4.06	0.99
9	27	7.9	110.6	9.10	3.72	1.07
9	21	6.2	84.3	5.56	3.70	0.95
8	22	6.5	71 9	6.62	3.33	1.01
8	18	5.3	57.8	4.35	3.30	0.91
7	20	5.9	49.7	5.52	2.91	0.97
7	15.5	4.6	38.6	3.47	2.91	0.87
6	16	4.7	28.6	3.24	2.47	0.83
6	13	3.8	23.5	2.27	2.48	0.77
5	13	3.8	15.7	1.99	2.03	0.72
5	10	3.0	12.4	1.29	2.05	0.66
4	10	2.9	7.7	1.22	1.62	0.65
4	7.5	2.2	5.9	0.75	1.63	0.58

MOMENTS OF INERTIA AND RADII OF GYRATION OF CARNEGIE DECK BEAMS—IRON.

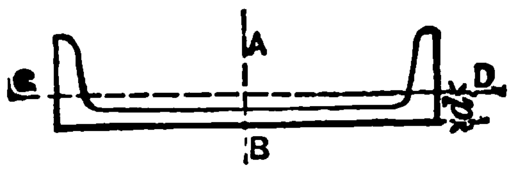


Size, in inches.	Weight per foot, in lbs.	I. Area of cross-section, in sq. in.	II. Moments of inertia. Axis A B.	III. Moments of inertia. Axis C D.	IV. Radii of gyration. Axis A B.	V. Radii of gyration. Axis C D.
10	26.9	8.1	118.4	6.12	3.83	0.87
10	35.2	10.6	139.9	7.41	3.64	0.84
9	23.2	7.0	77.6	2.45	3.34	0.59
9	29.8	8.9	91.0	3.15	3.19	0.59
8	21.4	6.4	52.1	2.23	2.85	0.59
8	28.0	8.4	63.2	2.96	2.74	0.59
7	17.0	5.1	34.4	1.81	2.60	0.59
7	22.8	6.9	41.8	2.34	2.47	0.58

DECK BEAMS—STEEL.

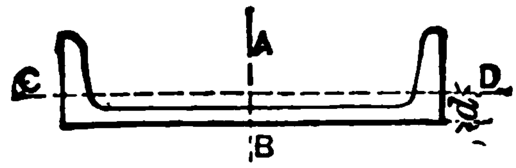
9	26	7.6	85.2	4.61	3.35	0.76
9	30	8.8	93.2	5.18	3.25	0.75
8	20	5.9	57.3	4.45	3.12	0.82
8	23.8	7.0	63.5	5.21	3.01	0.82
7	20	5.9	42.2	4.50	2.67	0.82
7	23.5	6.9	46.6	4.87	2.60	0.82

MOMENTS OF INERTIA AND RADII OF GYRATION OF CARNEGIE CHANNEL-BARS—IRON.



		I.	II.	IV.	VI.
Size, in inches.	Weight per foot, in lbs.	Area of cross-section, in sq. in.	Moments of inertia. Axis A B.	Radii of gyration. Axis A B.	Distance of centre of gravity from outside of web.
15	60	18	473.1	5.12	0.88
15	40	12	360.6	5.48	0.82
12	50	15	247.3	4.10	0.83
12	30	9	173.7	4.40	0.76
12	20	6	120.2	4.48	0.70
10	35	10.5	126.3	3.47	0.75
10	20	6.0	88.8	3.85	0.70
10	16	4.8	62.8	3.62	0.55
9	30	9.0	87.8	3.12	0.72
9	18	5.4	63.5	3.43	0.67
8	28	8.4	63.9	2.76	0.73
8	20	6.0	45.5	2.75	0.59
8	16	4.8	39.1	2.85	0.57
8	10	3.0	28.3	3.07	0.50
7	20	6.0	37.7	2.51	0.67
7	13½	4.0	25.5	2.51	0.52
7	8½	2.5	19.0	2.73	0.49
6	16	4.8	22.3	2.16	0.63
6	10	3.0	16.9	2.38	0.62
6	7½	2.2	12.2	2.34	0.48
5	14	4.2	13.10	1.77	0.61
5	8½	2.5	8.72	1.85	0.49
4	9	2.7	5.75	1.46	0.56
4	5	1.5	3.69	1.57	0.45
3½	8.1	2.4	3.82	1.25	0.52
3	6	1.8	2.22	1.15	0.51

MOMENTS OF INERTIA AND RADII OF GYRATION OF CARNEGIE CHANNEL-BARS—STEEL.



		I.	II.	IV.	VI.
Size, in inches.	Weight per foot, in lbs.	Area of cross-section, in sq. in.	Moments of inertia. Axis A B.	Radii of gyration. Axis A B.	Distance of centre of gravity from outside of web.
15	32	9.4	284.5	5.53	0.75
15	51	15.0	390.0	5.12	0.77
12	20	5.9	117.9	4.49	0.62
12	30¼	8.9	153.9	4.17	0.62
10	15¼	4.5	63.8	3.80	0.63
10	23¾	7.0	84.6	3.50	0.61
9	12¾	3.7	43.3	3.42	0.58
9	20½	6 0	58.5	3.14	0.56
8	10½	3.0	28.2	3.05	0.53
8	17¼	5.0	38.9	2.78	0.52
7	8½	2.5	17.4	2.67	0.49
7	14½	4.3	24.6	2.42	0.48
6	7	2.1	11.1	2.31	0.48
6	12	3.6	15.6	2.09	0.47
5	6	1.7	6.5	1.94	0 48
5	10½	3.0	9.1	1.75	0.47
4	5	1.4	3.5	1.57	0.48
4	8¼	2.4	4.8	1.81	0.48

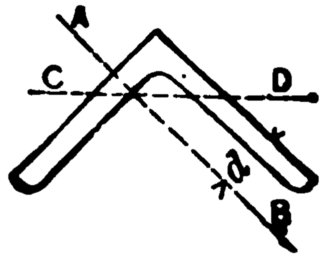
DECK BEAMS—STEEL.

9	26	7.6	85.2	3.35	....
9	30	8.8	93.2	3.25	....
8	20	5.9	57.3	3.12	....
8	23.8	7.0	63.5	3 01	....
7	20	5.9	42.2	2.67	....
7	23.5	6.9	46.6	2.60	....

MOMENTS OF INERTIA AND RADII OF GYRATION OF CARNEGIE ANGLE-BARS.

For minimum and maximum thicknesses and weight.

ANGLES WITH EQUAL LEGS—IRON OR STEEL.



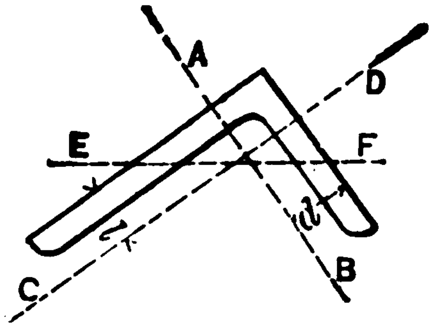
Weights in Table are for Iron; for Steel, add 2 per cent.

		I.	VI.	II.	IV.	V.
Size, in inches.	Weight, per foot.	Area of cross-section, in sq. in.	Distance of centre of gravity from outside of flange, in inches.	Moments of inertia. Axis A B.	Radii of gyration. Axis A B.	Radii of gyration. Axis C D.
6 × 6	16.9	5.06	1.66	17.68	1.87	1.19
	33.1	9.95	1.85	34.09	1.85	1.17
5 × 5	12.0	3.61	1.39	8.74	1.56	0.99
	27.6	8.28	1.61	20.00	1.55	1.00
4 × 4	9.5	2.86	1.14	4.36	1.23	0.79
	20.1	6.02	1.33	9.00	1.22	0.83
3½ × 3½	8.3	2.48	1.01	2.87	1.07	0.68
	17.4	5.22	1.20	5.90	1.06	0.72
3 × 3	4.8	1.44	0.84	1.24	0.93	0.58
	11.7	3.50	1.01	3.00	0.93	0.62
2¾ × 2¾	4.4	1.31	0.78	0.93	0.85	0.54
	9.0	2.69	0.95	2.22	0.91	0.66
2½ × 2½	4.0	1.19	0.72	0.70	0.77	0.50
	7.9	2.37	0 83	1.44	0.78	0.50
2¼ × 2¼	3.5	1.06	0.66	0.51	0.69	0.46
	7.0	2.11	0.78	1.04	0.70	0.49
2 × 2	2.4	0.71	0.57	0 28	0.62	0.40
	5.5	1.65	0.69	0.66	0.63	0.54
1¾ × 1¾	2.1	0.62	0.51	0.18	0.54	0.22
	4.9	1.47	0.64	0.44	0.55	0.40
1½ × 1½	1.8	0.53	0.44	0.11	0.46	0.29
	3.6	1 06	0.54	0.24	0 48	0.33
1¼ × 1¼	1.0	0.30	0.35	0.044	0.38	0.22
	1.9	0.56	0.40	0.077	0.37	0.24
1⅛ × 1⅛	0.9	0.27	0.32	0.032	0.34	0.19
	1.9	0.55	0.40	0.077	0.37	0.25
1 × 1	0.8	0.23	0.30	0.022	0.31	0.21
	1.5	0.44	0 34	0.037	0.29	0.18
¾ × ¾	0.6	0.17	0.23	0.009	0.23	0.14
	0.8	0.25	0.26	0.012	0.22	0.16

MOMENTS OF INERTIA AND RADII OF GYRATION OF CARNEGIE ANGLE-BARS.

For minimum and maximum thicknesses and weight.

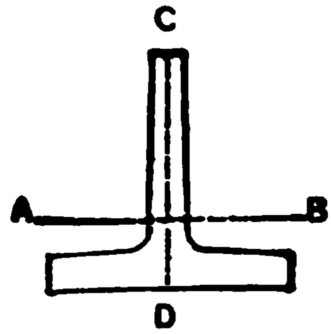
UNEVEN LEGS—IRON OR STEEL.



Weights in Table are for Iron; for Steel, add 2 per cent.

Size, in inches.	Weight, per foot.	I. Area of cross-section, in sq. in.	II. Moments of inertia. Axis A B.	III. Moments of inertia. Axis C D.	IV. Radii of gyration. Axis A B	V. Radii of gyration. Axis C D.	Radii of gyration. Axis E F.	VI. Distance from base to neutral axis. d.	VI. Distance from base to neutral axis. l.
6 × 4	12.0	3.61	13.47	4.90	1.93	1.17	.88	1.94	0.94
	27.3	8.18	29.58	10.68	1.90	1.14	.88	2.15	1.15
6 × 3½	11.4	3.42	12.86	3.34	1.94	0.99	.77	2.04	0.79
	25.8	7.75	28.20	7.25	1.91	0 97	.78	2.25	1.00
5 × 4	10.8	3.23	8.14	4.67	1.59	1.20	.86	1.53	1.03
	22.8	6.83	16.75	9.57	1.57	1.19	.88	1 72	1.22
5 × 3½	10.2	3.05	7.78	3.18	1.60	1.02	.76	1.61	0 86
	21.4	6.42	15.99	6.52	1.58	1.01	.77	1.80	1.05
5 × 3	9.5	2.86	7.37	2.04	1.61	0.85	.66	1.70	0.70
	20.1	6.02	15.19	4.18	1.59	0.83	.66	1.89	0.89
4½ × 3	8.9	2.67	5.50	1.98	1.44	0.86	.66	1.49	0.74
	18.7	5.62	11.26	4.06	1.42	0.85	.67	1.68	0.93
4 × 3½	8.9	2 67	4.18	2 99	1.25	1.06	.73	1.21	0.96
	18.7	5.61	8 53	6.10	1.23	1.04	.74	1.39	1.14
4 × 3	7.0	2.09	3.38	1.65	1.27	0.89	.65	1.26	0.76
	17.4	5.21	8.09	3.92	1.25	0.87	.66	1.47	0.97
3½ × 3	6.5	1.93	2.33	1.58	1.10	0.90	.63	1.06	0.81
	16.0	4.80	5.54	3.76	1.07	0.89	.65	1.27	1.02
3½ × 2½	4.8	1.44	1.80	0.78	1.12	0.74	.55	1.11	0.61
	9.8	2.92	4.08	1.84	1.17	0.78	.58	1.27	0.77
3¼ × 2	4.2	1.25	1.36	0.40	1.04	0.57	.44	1.09	0.48
	8.3	2.48	2.70	0.81	1.04	0.57	.45	1.22	0.59
3 × 2½	4.4	1.31	1.17	0.74	0.95	0.75	.53	0.91	0.66
	8.7	2.60	2.34	1.49	0.95	0.76	.54	1 03	0.78
3 × 2	4.0	1.19	1.09	0.39	0.96	0.57	.44	0.99	0.49
	8.0	2.31	2.27	0.84	0.99	0.60	.47	1.12	0.63
2½ × 2	2.7	0 81	0.51	0.29	0.79	0.60	.43	0 76	0 51
	7.2	2.18	1.38	0.80	0.80	0.61	.44	0.87	0.67
2 × 1⅜	2.6	0.78	0.37	0.12	0.63	0.39	.30	0.69	0.37
	4.6	1.39	0.56	0.22	0.63	0.40	.31	0.79	0.47
1⅜ × 1	0.9	0.28	0 05	0.02	0.44	0.29	.22	0.44	0.26

MOMENTS OF INERTIA AND RADII OF GYRATION OF CARNEGIE T-BARS—IRON OR STEEL.



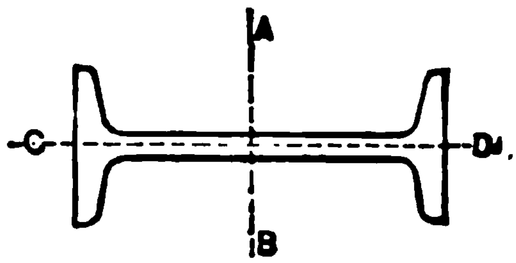
Weights in Table are for Iron; for Steel, add 2 per cent.

		I.	II.	III.	IV.	V.	VI.
Size, in inches.	Weight per foot.	Area of cross-section.	Moments of inertia. Axis A B.	Moments of inertia. Axis C D.	Radii of gyration. Axis A B.	Radii of gyration. Axis C D.	Distance d from base to neutral axis.
5 × 3	12.7	3.81	2.2	5.5	0.76	1.21	0.67
5 × 2½	10.4	3.12	1.3	4.9	0.64	1.26	0.57
4½ × 3½	15.5	4.65	5.1	3.7	1.04	0.90	1.11
4 × 5	15.2	4.56	10.7	2.8	1.54	0.79	1.56
4 × 5	11.8	3.54	8.5	2.1	1.56	0.78	1.51
4 × 4½	14.3	4.29	8.0	2.8	1.37	0.81	1.37
4 × 4½	11.2	3.36	6.3	2.1	1.38	0.80	1.31
4 × 4	12.0	3.60	5.1	2.5	1.20	0.83	1.15
4 × 3	9.1	2.73	2.0	2.1	0.86	0.88	0.78
4 × 2½	7.2	2.16	1.0	1.8	0.70	0.91	0.60
4 × 2	6.5	1.95	0.54	1.8	0.51	0.95	0.51
3½ × 4	12.5	3.75	5.5	1.89	1.21	0.72	1.25
3½ × 4	9.7	2.91	4.3	1.42	1.22	0.70	1.19
3½ × 3½	11.5	3.45	3.7	1.89	1.04	0.74	1.06
3½ × 3½	9.0	2.70	3.0	1.42	1.05	0.73	1.01
3½ × 3	10.7	3.21	2.4	1.88	0.87	0.77	0.88
3½ × 3	7.5	2.25	1.6	1.18	0.89	0.76	0.78
3 × 4	11.6	3.48	5.2	1.21	1.23	0.59	1.32
3 × 3½	10.7	3.21	3.5	1.20	1.06	0.62	1.12
3 × 3	9.8	2.94	2.3	1.20	0.88	0.64	0.93
3 × 3	6.5	1.95	1.6	0.75	0.90	0.62	0.86
3 × 2½	7.0	2.10	1.1	0.89	0.72	0.66	0.71
3 × 2½	6.0	1.80	0.94	0.75	0.73	0.65	0.68
2½ × 3	6.0	1.80	1.6	0.44	0.94	0.51	0.92
2½ × 2½	5.4	1.62	0.87	0.44	0.74	0.52	0.74
2¼ × 2¼	4.0	1.20	0.51	0.25	0.67	0.47	0.66
2 × 2	3.6	1.08	0.36	0.18	0.60	0.42	0.59
2 × 1½	3.0	0.90	0.16	0.18	0.42	0.45	0.42
1¾ × 1¾	3.0	0.90	0.23	0.12	0.51	0.37	0.54
1½ × 1½	2.5	0.75	0.15	0.08	0.49	0.34	0.42
1 × 1	1.2	0.36	0.03	0.02	0.29	0.21	0.33

MOMENTS OF INERTIA AND RADII OF GYRATION OF CARNEGIE Z-BARS—IRON OR STEEL.

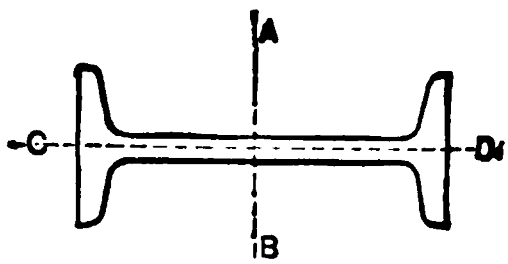
Weights in Table are for Iron; for Steel, add 2 per cent.

MOMENTS OF INERTIA AND RADII OF GYRATION OF TRENTON BEAMS—IRON.



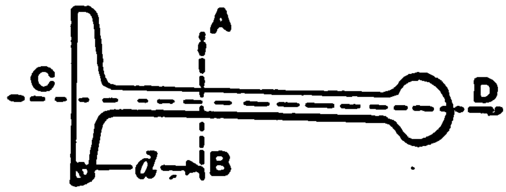
Size, in inches.	Weight per foot, in lbs.	I. Area of section, in sq. in.	II. Moments of inertia. Axis A B.	III. Moments of inertia. Axis C D.	IV. Radii of gyration. Axis A B.	V. Radii of gyration. Axis C D.
20	90.6	27.20	1,650.3	46.50	7.79	1.30
20	66.6	19.97	1,238.0	26.62	7 88	1.15
15	66.6	20.02	707.1	27.46	5.94	1.17
15	50	15.04	523.5	15 29	5.90	1.01
15	41.6	12.36	434.5	11.64	5.98	1.02
12¼	56.6	16.77	391.2	25.41	4.83	1.23
12¼	41.6	12.33	288.0	11.54	4.80	.97
12	40	11.73	281.3	16.76	4.90	1.20
12	32	9.46	229.2	11.66	4.92	1.11
10½	45	13.36	233.7	15.80	4.18	1.10
10½	35	10.44	185.6	9.43	4.22	.96
10½	30	8.90	164.0	8.09	4.29	.95
9	41.6	12.33	150.8	11.28	3.47	.95
9	28.3	8.50	111.9	7.35	3.63	.93
9	23.3	7.00	93.9	4.92	3.66	.84
8	26.6	8.03	83.9	7.55	3.23	.98
8	21.6	6.37	67.4	4.55	3.24	.85
7	18.3	5.50	44.3	3.90	2.84	.84
6	40	11.84	64.9	18.59	2.35	1.25
6	30	8.70	49.8	10.78	2.39	1.11
6	16.6	4.97	29.2	2.86	2.42	.76
6	13.3	3.98	23.5	1.61	2.43	.64
5	13.3	3.90	15.4	1.68	1.94	.66
5	10	2.99	12.1	1.04	1.99	.59
4	12.3	3 66	9.2	1.74	1.59	.69
4	10	2.91	7.5	1.11	1.60	.62
4	6	1.77	4.5	.31	1.60	.48

)MENTS OF INERTIA AND RADII OF GYRATION OF TRENTON BEAMS—STEEL.



		I.	II.	III.	IV.	V.
Size, in inches.	Weight per foot, in lbs.	Area of cross-section, in sq. in.	Moments of inertia.		Radii of gyration.	
			Axis A B.	Axis C D.	Axis A B.	Axis C D.
15	50	14.70	529.7	20.96	6.00	1.19
15	41	12.02	424.4	13.94	5.94	1.07
12	40	11.73	281.3	16.76	4.89	1.19
12	32	9.46	229.2	11.64	4.92	1.10
10	45	13.14	216.1	17.94	4.05	1.17
10	33	9.67	161.3	11.81	4.08	1.10
10	25.3	7.50	123.6	7.32	4.06	.98
9	27	7.98	110.6	9.13	3.72	1.07
9	21	6.15	84.3	5.56	3.70	.95
8	22	6.47	71.9	6.62	3.34	1.01
8	18	5.28	57.7	4.36	3.30	.91
7	20	5.87	49.7	5.51	2.91	.97
7	15.5	4.55	38.6	3.47	2.91	.87
6	16.6	4 97	29.2	2.86	2.42	.76
6	13.3	3.97	23.4	1.62	2.42	.64
5	13	3.80	15.7	1.98	2.03	.72
5	10	2.96	12.4	1.30	2 04	.67
4	10	2.94	7.7	1.22	1.62	.64
4	7.3	2.21	5.9	.75	1.63	.59

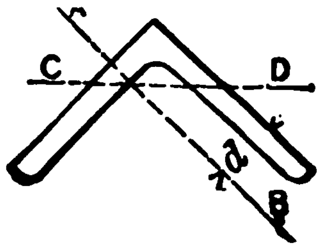
MOMENTS OF INERTIA AND RADII OF GYRATION OF TRENTON CHANNEL AND DECK BEAMS—IRON.



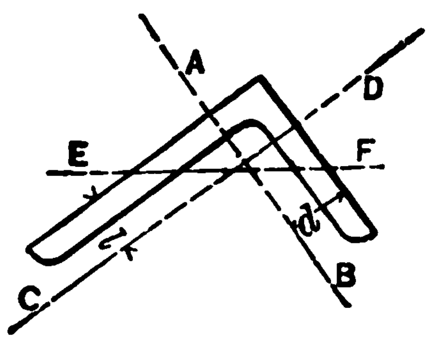
		I.	II.	III.	IV.	V.	VI.
Size, in inches.	Weight per foot, lbs.	Area of cross-section, sq. in.	Moments of inertia.		Radii of gyration.		Distance d of centre of gravity from outside of web.
			Axis A B.	Axis C D.	Axis A B.	Axis C D.	
Channel Bars.							
15	63.3	18.85	586.0	32.25	5.57	1.31	1.26
15	40	12.00	376.0	14.47	5.60	1.10	0.95
12¼	46.6	14.10	291.6	17.87	4.55	1.12	1.120
12¼	23.3	7.00	153.2	5.04	4.68	.85	0.755
10½	20	6.00	88.4	3.84	3.84	.80	0.628
10	16	4.77	64.0	2.20	3.68	.68	0.565
9	23.3	7.02	82.1	5.35	3 42	.87	0.85
9	16.6	5.08	58.8	2.53	3.40	.70	0.68
8	15	4.48	44.5	2.54	3.15	.75	0.76
8	11	3 30	32 9	1.44	3.16	.66	0.58
7	12	3.60	27.1	1.96	2.74	.88	0.715
7	8.5	2.54	17.3	.83	2.61	.57	0.511
6	15	4 32	21.7	2.12	2.24	.70	0.725
6	11	3.20	17.2	1.30	2 32	.64	0.68
6	7.5	2.25	12.6	.70	2.37	.55	0.54
5	6.3	1.92	7.2	.44	1.93	.48	0.464
4	5.5	1.65	3.9	.32	1.54	.44	0 46
3	5	1.45	2.0	.29	1.17	.45	0.51
Deck Beams.							
8	21.6	6.25	54.7	3.7	2.95	.76	......
7	18.3	5.35	35.1	3.6	2.56	.82	......

MOMENTS OF INERTIA OF TRENTON ANGLE-BARS.

		I.	II.	VI.	
Size, in inches.	Weight per foot, in lbs.	Area of cross-section, in sq. ins.	Moment of inertia.	Distance d from base to neutral axis, in inches.	

EVEN  LEGS.

6 in. × 6 in.	19 to 32½	5.75	19.910	1.685	Axis A B
4½ " × 4½ "	12½ to 20½	3.75	7.200	1.286	" "
4 " × 4 "	9½ to 18	2.86	4.360	1.138	" "
3½ " × 3½ "	8¼ to 14½	2.48	2.860	1.013	" "
3 " × 3 "	4.8 to 12¼	1.44	1.240	0.842	" "
2¾ " × 2¾ "	5.4 to 9½	1.62	1.150	0.802	" "
2½ " × 2½ "	3.9 to 7½	1.19	0.700	0.717	" "
2¼ " × 2¼ "	3½ to 6	1.06	0.500	0.654	" "
2 " × 2 "	3⅛ to 4½	0.94	0.350	0.592	" "
1¾ " × 1¾ "	2 to 3½	0.62	0.180	0.507	" "
1½ " × 1½ "	1¾ to 2¾	0.53	0.110	0.444	" "
1¼ " × 1¼ "	1 to 1⅞	0.30	0.044	0.358	" "
1 " × 1 "	¾ to 1½	0.23	0.022	0.296	" "
⅞ " × ⅞ "	0.6 to 1	0.20	0.014	0.264	" "
¾ " × ¾ "	9/16 to 0.8	0.17	0.009	0.233	" "

UNEVEN  LEGS.

6 in. × 4 in.	14 to 23	4.18	15.460	1.964	Axis C D
			5.600	0.964	" A B
5 " × 3½ "	10.2 to 19½	3.05	7.780	1.610	" C D
			3.190	0.860	" A B
4½ " × 3 "	9 to 14½	2.67	5.490	1.490	" C D
			1.980	0.740	" A B
4 " × 3 "	7 to 14½	2.09	3.370	1.260	" C D
			1.640	0.760	" A B
3½ " × 1½ "	4.0	1.19	1.500	1.320	" C D
			0.170	0.320	" A B
3 " × 2½ "	4⅜ to 9½	1.31	1.170	0.910	" C D
			0.740	0.660	" A B
3 " × 2 "	4 to 7½	1.19	1.090	0.990	" C D
			0.390	0.490	" A B

MOMENTS OF INERTIA OF TRENTON T-BARS.

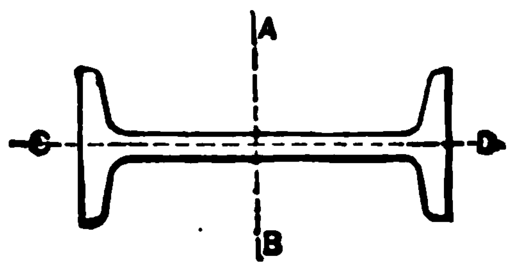
		I.	II.	IV.	VI.	
Size, in inches.	Weight per foot, in lbs.	Area of cross-section, in sq. in.	Moment of inertia.	Radii of gyration.	Distance d from base to neutral axis, in inches.	
4* × 4	12½	3.75	5.560 2.620	1.22 .84	1.180	Axis A B. Axis C D.
3½ × 3½	9.6	2.87	3.260 1.530	1.06 .73	1.030	Axis A B. Axis C D.
3 × 3	7	2.11	1.760 0.970	.91 .62	0.890	Axis A B. Axis C D.
2½ × 2½	5	1.46	0.850 0.400	.76 .52	0.740	Axis A B. Axis C D.
2 × 2	3⅛	0.94	0.350 0.160	.60 .42	0.590	Axis A B. Axis C D.
5 × 2½	11.7	3.50	1.500 5.090	.65 1.20	0.610	Axis A B. Axis C D.
3 × 2	4.8	1.45	0.470 0.680	.57 .68	0.520	Axis A B. Axis C D.
2 × 1½	3.00	0.91	0.170 0.180	.43 .45	0.500	Axis A B. Axis C D.
2¼ × 1⅛	2.40	0.74	0.060 0.180	.29 .49	0.290	Axis A B. Axis C D.
2 × 1	2.15	0.65	0.040 0.140	.26 .46	0.260	Axis A B. Axis C D.
[illegible] × 1	1.86	0.56	0.040 0.070	.26 .35	0.280	Axis A B. Axis C D.

* The first dimension is the width.

TRENTON IRON OR STEEL Z-BARS.

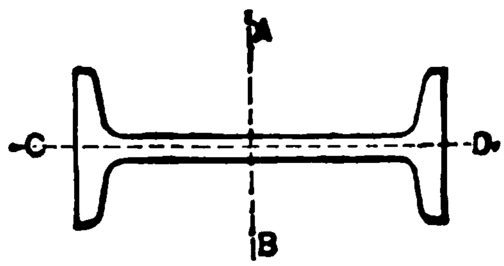
PHŒNIX IRON Z-BARS.

MOMENTS OF INERTIA AND RADII OF GYRATION OF JONES & LAUGHLIN'S, LIMITED, STEEL BEAMS.



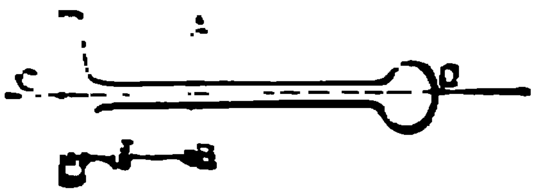
		I.	II.	III.	IV.	V.
Size, in inches.	Weight per foot, in lbs.	Area of cross-section, in sq. in.	Moments of Inertia.		Radii of Gyration.	
			Axis A B.	Axis C D.	Axis A B.	Axis C D.
15	70	20.6	731.1	37.8	5.95	1.35
15	59	17.3	640.9	30.3	6.08	1.33
15	48	14.1	495.9	19.2	5.93	1.16
15	39	11.5	403.3	13.1	5 92	1.06
12	50	14.7	302.0	18.1	4.53	1.11
12	38	11.2	265 4	15.6	4.86	1.18
12	30	9.1	211.7	10.2	4.82	1.05
10	32	9.4	152.6	10.8	4.02	1.07
10	23.8	7.0	117.7	7.09	3.88	.95
9	24.5	7.2	101.1	7.80	3.74	1.04
9	19.75	5.8	79.8	5.03	3.71	0.93
8	25	7.3	71.8	6.66	3.13	0 95
8	18	5.3	57.3	4.27	3.28	0.89
7	18.8	5.4	46.4	5.02	2.93	0.96
7	15.25	4.5	37.9	3.38	2.89	0.86
6	16.6	4.9	28.4	3.39	2.40	0.83
6	12.75	3.7	23.1	2.22	2.49	0.77
5	13	3.8	15.7	1.83	2.02	0.69
5	10	2.9	13.5	1.40	2.15	0.69
4	10.2	3.0	7.7	1.20	1.42	0.55
4	6.85	2.0	5.8	0.71	1.70	0.59
3	7	2.0	3.1	0 65	1 24	0.56
3	5.1	1.5	2.3	0.35	1.23	0.47

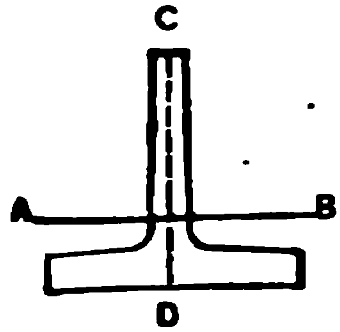
MOMENTS OF INERTIA AND RADII OF GYRATION OF PHŒNIX BEAMS—STEEL.



Size, in inches.	Weight per foot, in lbs.	I. Area of cross-section, in sq. in.	II. Moments of inertia. Axis A B.	III. Moments of inertia. Axis C D.	IV. Radii of gyration. Axis A B.	V. Radii of gyration. Axis C D.
15	75	22.05	757.7	40.1	5.86	1.35
15	60	17.64	644.0	30.4	6.04	1.32
15	50	14.70	529.7	21.0	6.00	1.20
15	41	12 05	424.1	14.0	5.94	1.08
12	40	11.76	281.3	16.8	4.90	1.20
12	32	9.41	222.3	10.3	4.85	1.04
10½	33	9.70	179.6	11.8	4.54	1.10
10½	25½	7.47	137.3	7.32	4.52	0.99
9	27	7.93	110.6	9.10	3.72	1.07
9	21	6.17	84.3	5.56	3.70	0.95
8	22	6.47	71.9	6.62	3.38	1.01
8	18	5.29	57.8	4.35	3.30	0.91
7	20	5.88	49.7	5.52	2.91	0.97
7	15½	4.55	38.6	3.47	2.91	0.87
6	16	4.70	28.6	3.24	2.47	0.83
6	13	3.82	23.5	2.27	2.48	0.77
5	13	3.82	15.7	1.99	2.03	0.72
5	10	2.94	12.4	1.29	2.05	0.66
4	10	2.94	7.7	1.22	1.62	0.65

MOMENTS OF INERTIA AND RADII OF GYRATION OF PHŒNIX DECK-BEAMS AND T-BARS.

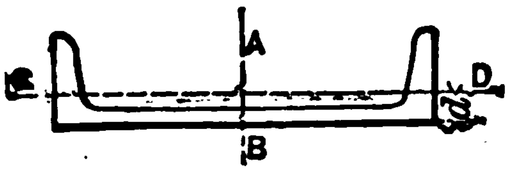




Size, in inches.	Weight per foot, in lbs.	I. Area of cross-section, in sq. in.	II. Moments of inertia. Axis A B.	III. Moments of inertia. Axis C D.	IV. Radii of gyration. Axis A B.	V. Radii of gyration. Axis C D.	VI. Distance d from base to neutral axis.
Deck Beams—Iron.							
[illegible]	[illegible]	[illegible]	[illegible]	5.17	4.21	0.74	4.27
[illegible]	[illegible]	[illegible]	[illegible]	5.16	4.22	0.78	3.77
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.27	0.84	2.96
[illegible]	[illegible]	[illegible]	[illegible]	3.55	2.90	0.80	2.59
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.53	0.77	2.25
[illegible]	[illegible]	[illegible]	[illegible]	2.35	2.17	0.75	1.88
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.79	0.51	2.41
Deck Beams—Steel.							
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3.31	0.69	4.08
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.92	0.73	3.62
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.60	0.85	2.85
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.18	0.76	2.39
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.16	0.73	2.78
T-Bars—Iron.							
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.22	0.77
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.17	0.66
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.98	0.76
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.01	0.57
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.58	0.84
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.76	1.02
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.69	0.89
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.53	0.75
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.14	0.63

[illegible] dimension is the width, [illegible] second the height.

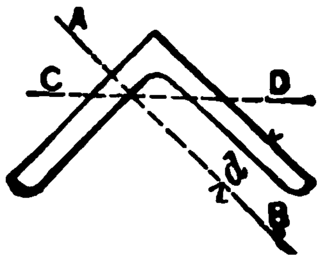
MOMENTS OF INERTIA AND RADII OF GYRATION OF PHŒNIX CHANNEL-BARS—IRON.



		I.	II.	III.	IV.	V.	VI.
Size, in inches.	Weight per foot, in lbs.	Area of cross-section, in sq. in.	Moments of inertia.		Radii of gyration.		Distance *d* from base to neutral axis.
			Axis A B.	Axis C D.	Axis A B.	Axis C D.	
15	66.6	20	554.57	23.61	5.27	1.09	1.08
15	50	15	421.87	12.39	5.30	0.91	0.86
15	38.3	11.5	351.56	10.01	5.53	0.93	0.83
12	50	15.0	235.73	8.44	3.96	0.75	0.80
12	29.3	8.8	159.44	4.19	4.26	0.69	0.82
12	20	6.0	123.50	3.01	4.54	0.71	0.86
10	37	11.1	128.61	5.26	3.40	0.69	0.76
10	25	7.5	97.36	3.51	3.60	0.69	0.66
10	16	4.8	63.67	2.21	3.64	0.68	0.56
9	33.3	10.0	94.27	5.24	3.07	0.72	0.76
9	23.3	7.0	75.29	3.69	3.28	0.73	0.70
9	15	4.5	61.01	2.36	3.49	0.69	0.70
8	19	5.7	43.99	2.14	2.76	0.61	0.56
8	10	3.0	26.20	0.85	2.96	0.53	0.45
7	19	5.7	32.69	2.00	2.40	0.59	0.59
7	8.3	2.5	17.62	0.75	2.66	0.55	0.47
6	15.6	4.7	23.12	2.5	2.22	0.73	0.73
6	7.3	2.2	10.42	0.62	2.18	0.53	0.40
5	9	2.7	9.52	0.84	1.88	0.56	0.55
5	5.6	1.7	6.35	0.43	1.93	0.51	0.47
4	8	2.4	5.53	0.79	1.52	0.57	0.60
4	5	1.5	3.74	0.4	1.58	0.52	0.52
3	6	1.8	2.26	0.36	1.12	0.45	0.53
3	5	1.5	1.98	0.29	1.15	0.44	0.50

MOMENTS OF INERTIA AND RADII OF GYRATION OF PHŒNIX ANGLE-BARS—IRON.

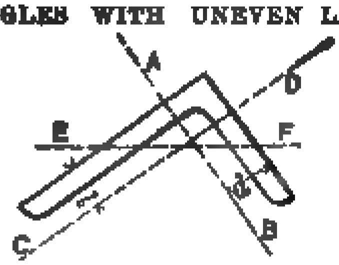
ANGLES WITH EQUAL LEGS.



		I.	II.	III.	IV.	V.	VI.
Size, in inches.	Weight per foot, in lbs.	Area of cross-section, sq. in.	Moments of inertia.		Radii of gyration.		Distance d from base to neutral axis.
			Axis A B.	Axis C D.	Axis A B.	Axis C D.	
6 × 6	33.3	10	35.17	13.98	1.87	1.18	1.84
6 × 6	16.8	5.03	17.22	6.77	1.85	1.16	1.58
5 × 5	20.6	6.2	14.70	6.07	1.54	0.99	1.55
5 × 5	12.3	3.7	9.35	3.77	1.59	1.01	1.46
4 × 4	17.2	5.16	7.18	3.01	1.18	0.76	1.22
4 × 4	9.4	2.81	4.39	1.71	1.25	0.78	1.16
3½ × 3½	13.6	4.1	4.35	1.84	1.03	0.67	1.06
3½ × 3½	6.8	2.05	2.30	0.95	1.06	0.68	0.98
3 × 3	9.4	2.81	2.23	0.95	0.89	0.58	0.98
3 × 3	5	1.5	1.33	0.54	0.94	0.6	0.87
2¾ × 2¾	8.6	2.58	1.65	0.62	0.80	0.49	0 88
2¾ × 2¾	4.5	1.34	1.01	0.41	0.87	0.55	0.82
2½ × 2½	7.9	2.36	1.22	0.52	0.72	0.47	0.77
2½ × 2½	3.5	1.05	0.62	0.25	0.77	0.49	0.7
2¼ × 2¼	6.1	1.83	0.82	0.35	0.67	0.44	0.74
2¼ × 2¼	2.6	0.8	0.40	0.17	0.71	0.46	0.69
2 × 2	4.6	1.4	0.49	0.20	0.59	0.38	0.62
2 × 2	2.5	0.75	0.29	0.12	0.62	0.40	0.6
1¾ × 1¾	2.0	0.61	0.18	0.07	0.55	0.35	0.52
1½ × 1½	1.5	0.44	0.9	0.04	0.46	0.29	0.44

MOMENTS OF INERTIA AND RADII OF GYRATION OF PHŒNIX ANGLE-BARS—IRON.

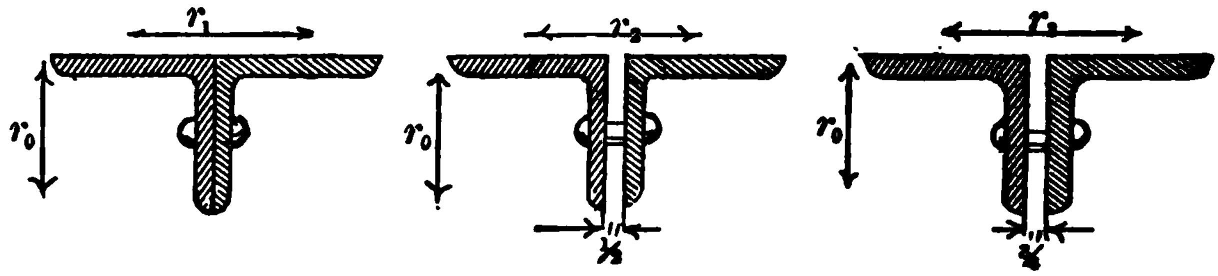
ANGLES WITH UNEVEN LEGS.



NOTE.—E F is parallel to line through ends of sides.

RADII OF GYRATION FOR A PAIR OF CARNEGIE ANGLES PLACED BACK TO BACK.

ANGLES WITH EQUAL LEGS.

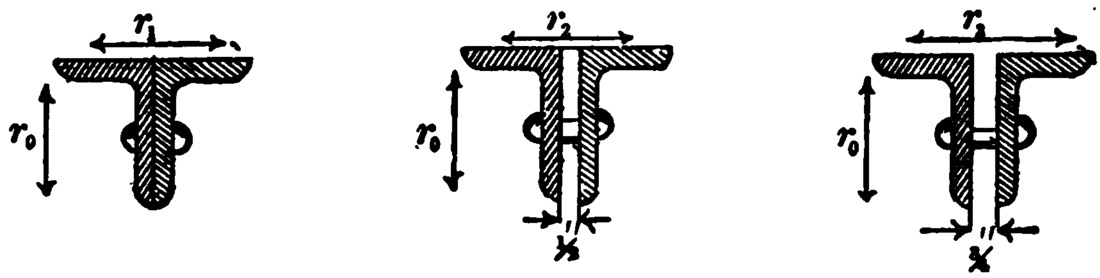


Radii of Gyration given, correspond to directions indicated by arrow-heads.

Size, in inches.	*Area of cross-section, in inches.	Weight per foot of single angle, in lbs.	Radii of gyration. r_0.	r_1.	r_2.	r_3.
6 × 6	10.12	16.9	1.87	2.50	2.67	2.76
6 × 6	19.90	33.1	1.85	2.62	2.80	2.89
5 × 5	7.22	12.0	1.56	2.09	2.26	2.35
5 × 5	16.56	27.6	1.55	2.24	2.42	2.52
4 × 4	5.72	9.5	1.23	1.68	1.86	1.95
4 × 4	12.04	20.1	1.22	1.81	2.00	2.10
3½ × 3½	4.96	8.3	1.07	1.47	1.66	1.75
3½ × 3½	10.44	17.4	1.06	1.60	1.80	1.90
3 × 3	2.88	4.8	0.93	1.25	1.43	1.53
3 × 3	7.00	11.7	0.93	1.37	1.56	1.66
2¾ × 2¾	2.62	4.4	0.85	1.15	1.34	1.44
2¾ × 2¾	5.38	9.0	0.91	1.31	1.50	1.61
2½ × 2½	2.38	4.0	0.77	1.05	1.24	1.34
2½ × 2½	4.74	7.9	0.78	1.14	1.33	1.43
2¼ × 2¼	2.12	3.5	0.69	0.96	1.14	1.24
2¼ × 2¼	4.22	7.0	0.70	1.05	1.24	1.35

RADII OF GYRATION FOR A PAIR OF CARNEGIE ANGLES PLACED BACK TO BACK.

ANGLES WITH UNEQUAL LEGS.



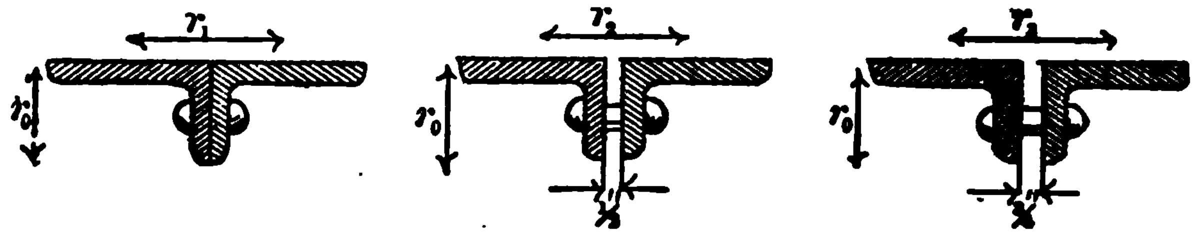
Radii of Gyration given, correspond to directions indicated by arrow-heads.

Size, in inches.	*Area of cross-section, in inches.	Weight per foot of single angle, in lbs.	Radii of gyration. r_0.	r_1.	r_2.	r_3.
6 × 4	7.22	12.0	1.93	1.50	1.67	1.76
6 × 4	16.36	27.3	1.90	1.62	1.80	1.90
6 × 3½	6.84	11.4	1.94	1.26	1.43	1.53
6 × 3½	14.50	25.8	1.91	1.39	1.58	1.68
5 × 4	6.46	10.8	1.59	1.58	1.75	1.85
5 × 4	13.66	22.8	1.57	1.70	1.89	1.98
5 × 3½	6.10	10.2	1.60	1.33	1.51	1.60
5 × 3½	12.84	21.4	1.58	1.45	1.64	1.74
5 × 3	5.72	9.5	1.61	1.10	1.27	1.37
5 × 3	12.04	20.1	1.59	1.22	1.41	1.51
4½ × 3	5.34	8.9	1.44	1.13	1.31	1.41
4½ × 3	11.24	18.7	1.42	1.26	1.45	1.56
4 × 3½	5.34	8.9	1.25	1.48	1.60	1.70
4 × 3½	11.22	18.7	1.23	1.54	1.74	1.84
4 × 3	4.18	7.0	1.27	1.17	1.35	1.44
4 × 3	10.42	17.4	1.25	1.30	1.50	1.60
3½ × 3	3.86	6.5	1.10	1.22	1.40	1.49
3½ × 3	9.60	16.0	1.07	1.35	1.55	1.65
3½ × 2½	2.88	4.8	1.12	0.96	1.13	1.23
3½ × 2½	5.94	9.8	1.17	1.10	1.28	1.39
3¼ × 2	2.50	4.2	1.04	0.74	0.92	1.02
3¼ × 2	4.96	8.3	1.04	0.82	1.02	1.12
3 × 2½	2.62	4.4	.95	1.00	1.18	1.28
3 × 2½	5.20	8.7	.95	1.09	1.28	1.38
3 × 2	2.38	4.0	.96	0.75	0.93	1.03
3 × 2	4.62	8.0	.99	0.87	1.06	1.17
2½ × 2	1.62	2.7	.79	0.79	0.97	1.07
2½ × 2	4.36	7.2	.80	0.90	1.10	1.21

* The figures in this column give the area of both angles.

RADII OF GYRATION FOR A PAIR OF CARNEGIE ANGLES PLACED BACK TO BACK.

ANGLES WITH UNEQUAL LEGS.



Radii of Gyration given, correspond to directions indicated by arrow-heads.

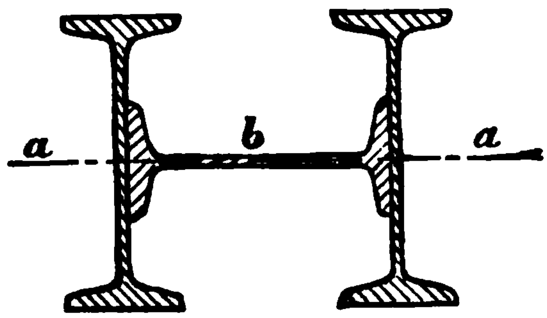
Size, in inches.	*Area of cross-section, in inches.	Weight per foot of single angle, in lbs.	Radii of gyration. r_0.	r_1.	r_2.	r_3.
6 × 4	7.22	12.0	1.17	2.74	2.92	3.01
6 × 4	16.36	27.3	1.14	2.87	3.06	3.16
6 × 3½	6.84	11.4	0.99	2.81	3.00	3.10
6 × 3½	14.50	25.8	0.97	2.95	3.14	3.24
5 × 4	6.46	10.8	1.20	2.20	2.38	2.48
5 × 4	13.66	22.8	1.19	2.33	2.52	2.62
5 × 3½	6.10	10.2	1.02	2.27	2.45	2.55
5 × 3½	12.84	21.4	1.01	2.39	2.59	2.69
5 × 3	5.72	9.5	0.85	2.35	2.52	2.62
5 × 3	12.04	20.1	0.83	2.47	2.66	2.77
4½ × 3	5.34	8.9	0.86	2.07	2.25	2.35
4½ × 3	11.24	18.7	0.85	2.20	2.39	2.49
4 × 3½	5.34	8.9	1.06	1.74	1.92	2.02
4 × 3½	11.22	18.7	1.04	1.86	2.05	2.15
4 × 3	4.18	7.0	0.89	1.79	1.97	2.07
4 × 3	10.42	17.4	0.87	1.93	2.12	2.22
3½ × 3	3.85	6.5	0.90	1.52	1.71	1.80
3½ × 3	9.60	16.0	0.89	1.66	1.86	1.96
3½ × 2½	2.88	4.8	0.74	1.58	1.76	1.86
3½ × 2½	5.94	9.8	0.78	1.72	1.91	2.01
3¼ × 2	2.50	4.2	0.57	1.51	1.70	1.80
3¼ × 2	4.96	8.3	0.57	1.60	1.80	1.91
3 × 2½	2.62	4.4	0.75	1.31	1.50	1.59
3 × 2½	5.20	8.7	0.76	1.40	1.59	1.69
3 × 2	2.38	4.0	0.57	1.38	1.57	1.67
3 × 2	4.62	8.0	0.60	1.49	1.69	1.79
2½ × 2	1.62	2.7	0.60	1.10	1.28	1.39
2½ × 2	4.36	7.2	0.61	1.18	1.37	1.48

* The figures in this column give the area of both angles.

For compound sections made up of two or more beams or bars, the moments of inertia are found by combining those of the several shapes as given in the preceding tables. Thus: —

I = Twice the moment of inertia for beam a (col. II.) + that for beam b (col. III.).

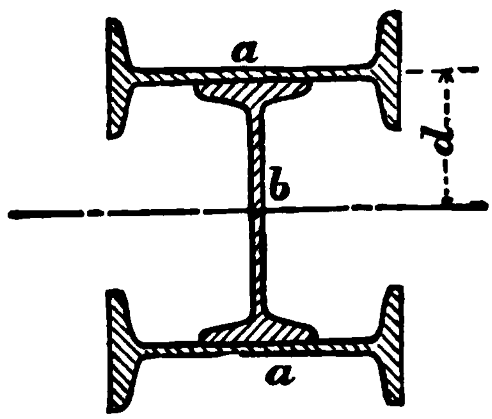
$$G^2 = \frac{I}{\text{sum of areas of beams } a \text{ and } b \text{ (col. I.)}}.$$



I = Twice area of beam a (col. I.) $\times d^2$ + twice moment of inertia for beam a (col. III.) + that for beam b (col. II.).

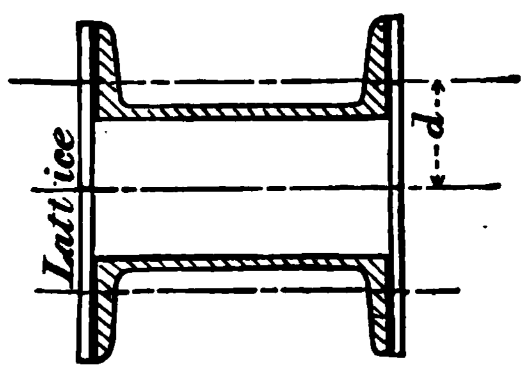
$$R = \frac{I}{d + \frac{1}{2} \text{ width flange of beam } a}.$$

$$G^2 = \frac{I.}{\text{sum of areas of beams } a \text{ and } b \text{ (col. I.)}}$$



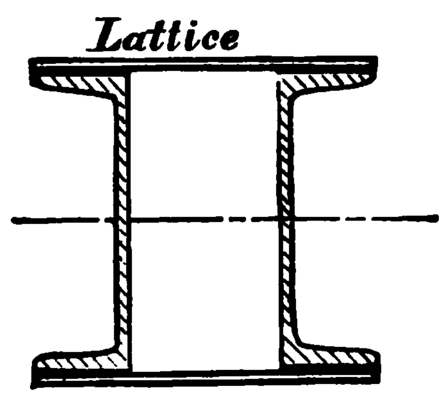
I = Twice area of channels (col. I.) $\times d^2$ + moment of inertia (col. III.), in which d = distance of centre of gravity of the channel from centre line of the combination.

$$G^2 = \frac{I}{\text{area of the two channels (col. I.)}}.$$



I = Twice the moment in col. II.

G^2 = Same as for single channel.



When a section is employed alone, either as girder or post, the neutral axis passes through its centre of gravity. When rigidly connected with other sections forming part of a compound section, the neutral axis passes through the centre of gravity of the com-

pound section; and therefore the moment of inertia of the elementary section will not be that around its own centre of gravity, but around an axis at a distance from that point. *The moment of inertia of a section about an axis other than that through its centre of gravity* is equal to the moment about a parallel axis passing through its centre of gravity plus the area of the section multiplied by the square of the distance between the axes.

The first step, then, in finding the moment of inertia, is to find the position of the centre of gravity of the section. For all symmetrical sections, this, of course, lies at the middle of the depth. For triangles, it is found on a line parallel with the base, and distant one-third the height of the triangle above the base. For other sections, it is found by supposing the area divided up into elementary sections, and multiplying the area of each such section by the distance of its centre of gravity from any convenient line. The sum of these products divided by the total area of the section will give the distance of the centre of gravity from the line from which the distances were measured.

EXAMPLE. — Find the neutral axis of a ⊥ section having the following dimensions: width, 8 inches; depth, 10 inches; thickness of metal, 2 inches. The area of the vertical flange, considering it as running through to the bottom of the section, would be 10 × 2, or 20 square inches; and the distance of its centre of gravity above the bottom line, 5 inches. The product of these quantities, therefore, is 100. The area of the bottom flange, not included in the vertical flange as above taken, is 6 times 2, or 12 square inches; the distance of its centre of gravity above the bottom line, 1 inch; and the product of the two, therefore, 12. The sum of these products divided by the total area is $\frac{112}{32}$, or 3.5 inches, which is the distance of the centre of gravity above the bottom line of the section.

Having found the neutral axis of this section, its moment of inertia is readily found by the formula before given. Thus, in the case just supposed, d would be $10 - 3.5 = 6.5$, $d_{,} = 3.5$; $d_{,,} = 1.5$; and the moment would be (see p. 299),

$$I = \frac{(2 \times 6.5^3) + (8 \times 3.5^3) - (6 \times 1.5^3)}{3} = 290\tfrac{2}{3}.$$

The moment of resistance of this section as a girder would be $\frac{290.3}{6.5}$, or 44; and if a strain on the fibres of the iron of 12,000 pounds per square inch be allowed, then, since the moment of resistance of the girder multiplied by strain per square inch must

equal the bending-moment of the load, it will be able to support a load whose bending-moment is 44⅔ times 12,000 pounds, or 536,000; i.e., if used as a girder secured rigidly at one end, and loaded at the other, it would support a load, in pounds, of

$$\frac{536000}{\text{length in inches}}.$$

Or if supported at both ends, and the load uniformly distributed over the span, it would support a load eight times as great; the bending-moment in such case being one-eighth that in the former case (see pp. 291, 292).

NOTE.—The formulas and figures on pp. 298, 299, and 325, are taken, by permission of *The New-Jersey Steel and Iron Company*, from a hand-book which they publish, entitled "Useful Information for Engineers and Architects," and containing full information pertaining to the forms of iron which they manufacture.

Radius of Gyration of Compound Shapes.

(*Ninth Edition.*)

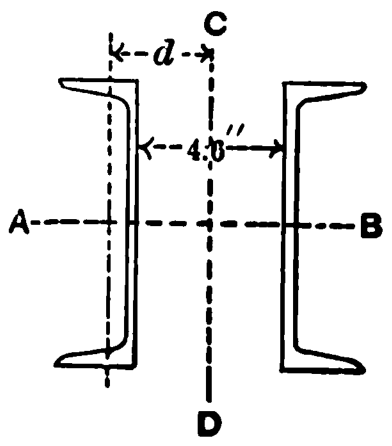
In the case of a pair of any shape without a web the value of R can always be readily found without considering the moment of inertia.

The radius of gyration for any section around an axis parallel to another axis passing through its centre of gravity, is found as follows:

Let r = radius of gyration around axis through centre of gravity; R = radius of gyration around another axis parallel to above; d = distance between axes.

$$R = \sqrt{d^2 + r^2}.$$

When r is small, R may be taken as equal to d without material error. Thus, in the case of a pair of channels latticed together, or a similar construction.



EXAMPLE 1.—Two 9-inch, 15-pound Phœnix channel bars are placed 4.6 inches apart, as in the figure; required the radius of gyration around axis $C\,D$ for combined section.

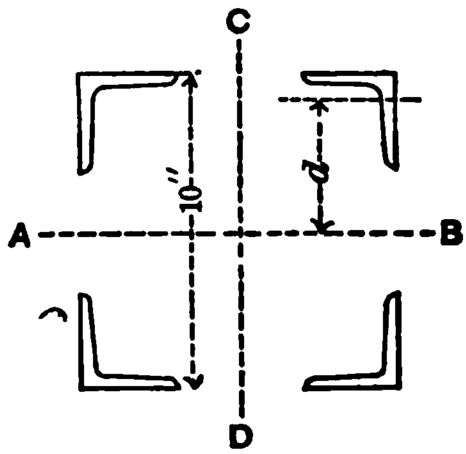
Ans. Find r, in Column V., p. 319 = 0.69; and r^2 = .4761.

Distance from base of channel to neutral axis, Column VI., is .7. One-half of 4.6 = 2.3 + .7 = 3, the distance between neutral axis of single channel and of combined section; hence,

$R = \sqrt{9 + .4761} = 3.077$; or, for all practical purposes, $R = d$.

EXAMPLE 2.—Four 3 × 3 inches, 5-pound Phœnix angles as shown form a column 10 inches square; find the rad gyration.

Ans. From Column IV., p. 320, we find $r = 0.94$ an



.8336. The distance from base of angle to neutral axis, C VI., is .87; hence, $d = 5 - .87 = 4.13$; or, $d^2 = 17.0569$
$R = \sqrt{17.0569 + .8336} = 4.23.$

CHAPTER XIV.

GENERAL PRINCIPLES OF THE STRENGTH OF BEAMS, AND STRENGTH OF IRON BEAMS.

BY the term "beam" is meant any piece of material which supports a load whose tendency is to break the piece across, or at right angles to, the fibres, and which also causes the piece to bend before breaking. When a load of any kind is applied to any beam, it will cause it to bend by a certain amount; and as it is impossible to bend a piece of any material without stretching the fibres on the outer side, and compressing the fibres on the inner side, the bending of the beam will produce tension in its lower fibres, and compression in its upper ones. This tension and compression are also greatest in those fibres which are the farthest from the *neutral axis* of the beam. The *neutral axis* is the line along which the fibres of the beam are neither lengthened nor shortened by the bending of the beam. For beams of wrought-iron and wood the neutral axis practically passes through the centre of gravity of the cross-section of the beam.

To determine the strength of any beam to resist the effects of any load, or series of loads, we must determine two things: first, the destructive force tending to bend and break the beam, which is called the "bending-moment;" and, second, the combined resistance of all the fibres of the beam to being broken, which is called the "moment of resistance."

The methods for finding the bending-moments for any load, or series of loads, have been given in Chap. XII.; and rules for finding the moment of resistance, which is equal to the moment of inertia divided by the distance of the most extended or compressed fibres from the neutral axis, and the quotient multiplied by the strength of the material, have been given in Chap. XIII., together with tables of the moment of inertia for rolled iron sections of the usual patterns.

Now, that a beam shall just be able to resist the load, and not break, we must have a condition where the bending-moment in the beam is equal to the moment of resistance multiplied by the strength of the material. That the beam may be abundantly *safe* to resist the given load, the moment of resistance multiplied by

strength of material must be several times as great as the bending-moment; and the ratio in which this product exceeds the bending-moment, or in which the breaking-load exceeds the safe load, is known as the "factor" of safety.

By "the strength of the material" is meant a certain constant quantity, which is determined by experiment, and which is known as the "Modulus of Rupture." Of course this value is different for each different material. The following table contains the values of this constant divided by the factor of safety, for most of the materials used in building-construction. The moment of resistance multiplied by these values will give the *safe resisting*-power of the beam.

Modulus of Rupture for Safe Strength.

Material.	Value of R. in lbs.	Material.	Value of R. in lbs.
Cast Iron	5,544	American white pine	1,080
Wrought Iron	12,000	American yellow pine	1,800
Steel	16,000	American spruce	1,250
		Oregon pine	1,350
American ash	2,000		
American red beech	1,800	Bluestone flagging (Hudson River)	375
American yellow birch	1,620	Granite, average	300
American white cedar	1,000	Limestone	270
American elm	1,400	Marble	300
New England fir	1,500	Sandstone	150
Hemlock	1,200	Slate	900
American white oak	1,350		

The above values of R for wrought iron and steel are one-fourth that for the breaking-loads; for cast-iron, one-sixth; for wood, one-third; and for stone, one-sixth. The constants for wood are based upon the recent tests made at the Massachusetts Institute of Technology upon full-size timbers of the usual quality found in buildings. The figures given in the above table are believed to be amply safe for beams in floors of dwellings, public halls, roofs, etc.; but, for floors in mills and warehouse-floors, the author recommends that not more than two-thirds of the above values be used. The safe loads for the Trenton, Phœnix, and Carnegie sections, used as beams, are all computed with 12,000 pounds for the safe value of R, or with 12,000 pounds fibre strain, as it is generally called, for iron, and 16,000 pounds for steel.

There are certain cases of beams which most frequently occur in building construction, for which formulas can be given by which the safe loads for the beams may be determined directly; but it often happens that we may have either a regularly shaped beam

irregularly loaded, or a beam of irregular section, but with a common method of loading, or both; and in such cases it is necessary to determine the bending-moment, or moment of resistance, and find the beam whose moment of resistance multiplied by R is equal to this bending-moment, or what load will give a bending-moment equal to the moment of resistance of a beam multiplied by R.

For example, suppose we have a rectangular beam of yellow pine loaded at irregular points with irregular loads: what dimensions shall the beam be to carry these loads? We will suppose that we have found the bending-moment caused by these loads to be 480,000 inch pounds.

Then, as bending-moment equals moment of resistance multiplied by R,

$$480{,}000 \text{ pounds} = \frac{B \times D^2}{6} \times 1800 = B \times D^2 \times 300;$$

or

$$B \times D^2 = \frac{480000}{300} = 1600.$$

If we assume $D = 12$ inches, then $B = \frac{1600}{144} = 11$ inches; or, the beam should be 11 inches by 12 inches.

If, instead of a hard-pine beam, we should wish to use an iron beam to carry our loads in the above example, we must find a beam whose moment of resistance multiplied by 12,000 equals 480,000 inch pounds. We can only do this by trial, and for the first trial we will take the Trenton 12¼-inch 125-pound beam. The moment of inertia of this beam is given as 288; and its moment of resistance is one-sixth of this, or 48. Multiplying this by 12,000, we have 576,000 pounds as the resisting-force of this beam, or 96,000 pounds over the bending-moment. Hence we should probably use this beam, as the next lightest beam would probably not be strong enough. In this way we can find the strength of a beam of any cross-section to carry any load, however irregularly disposed it may be.

Strength of Wrought-Iron Beams, Channels, Angle and T Bars.

It is very seldom that one needs to compute the strength of wrought-iron beams, channels, etc.; because, if he uses one of the regular sections to be found in the market, the computations have already been made by the manufacturers, and are given in their handbook. There might, however, be cases where it would be necessary to make the calculations for any particular beam; and to meet such cases we give the following formulas.

Beams fixed at one end, and loaded at the other (**Fig. 1**).

Safe load in pounds =

$$\frac{1000 \times \text{moment of inertia}}{\text{length in feet} \times y}. \qquad (1)$$

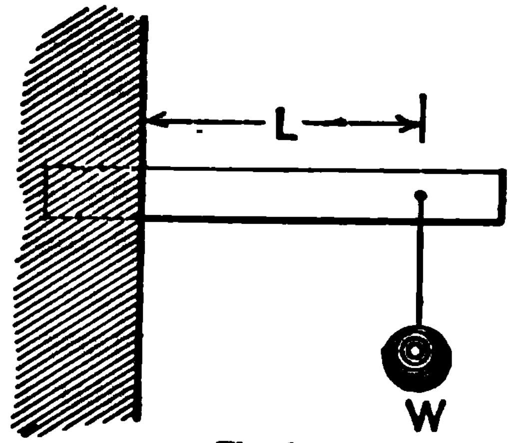


Fig. 1.

Beams fixed at one end, loaded with uniformly distributed load (Fig. 2).

Safe load in pounds =

$$\frac{2000 \times \text{moment of inertia}}{\text{length in feet} \times y}. \qquad (2)$$

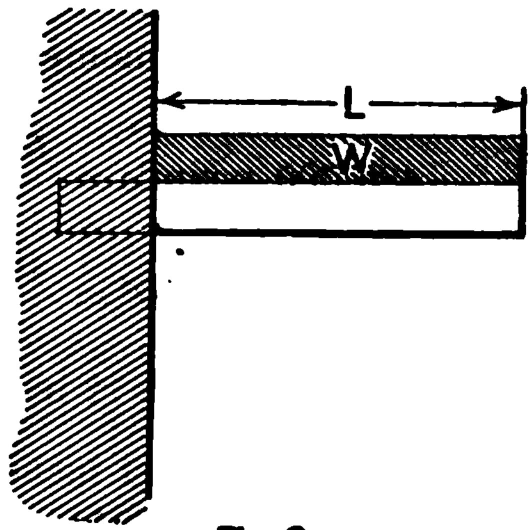


Fig. 2.

Beams supported at both ends, loaded at middle (**Fig. 3**).

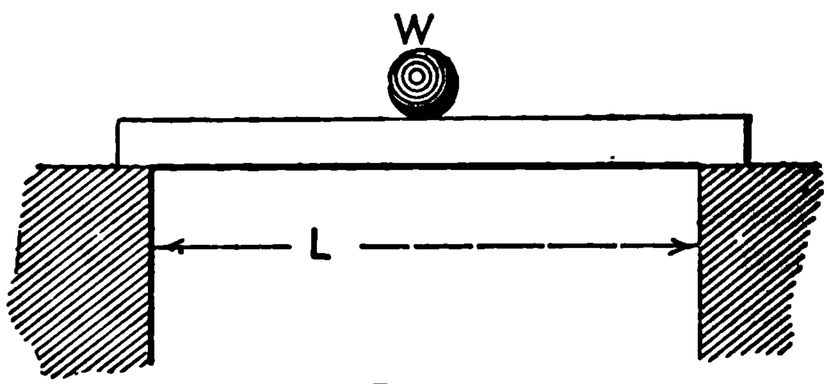


Fig. 3.

$$\text{Safe load in pounds} = \frac{4000 \times \text{moment of inertia}}{\text{span in feet} \times y}. \qquad (3)$$

Beams supported at both ends, load uniformly distributed (Fig. 4).

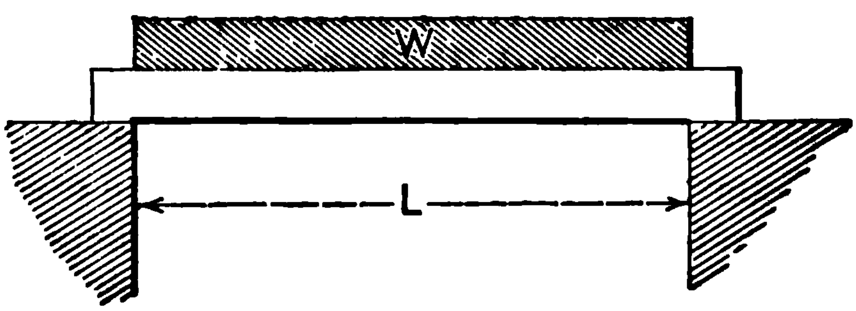


Fig. 4.

$$\text{Safe load in pounds} = \frac{8000 \times \text{moment of inertia}}{\text{span in feet} \times y}. \qquad (4)$$

Beams supported at both ends, loaded with concentrated load not at centre (Fig. 5).

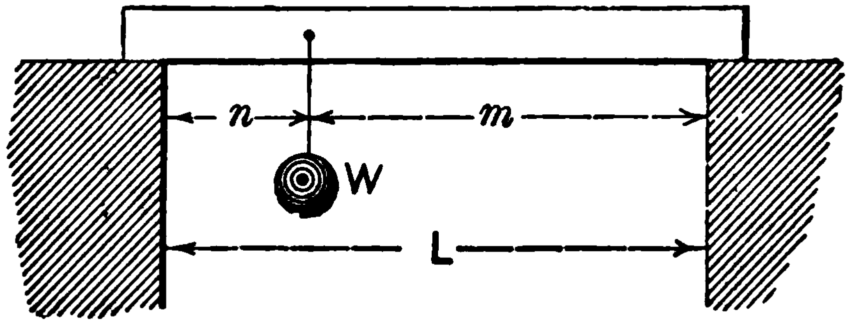


Fig. 5.

$$\text{Safe load in pounds} = \frac{1000 \times \text{moment of inertia} \times \text{span in feet}}{m \times n \times y}. \quad (5)$$

Beams supported at both ends, loaded with* W *pounds, at a distance* m *from each end (Fig. 6).

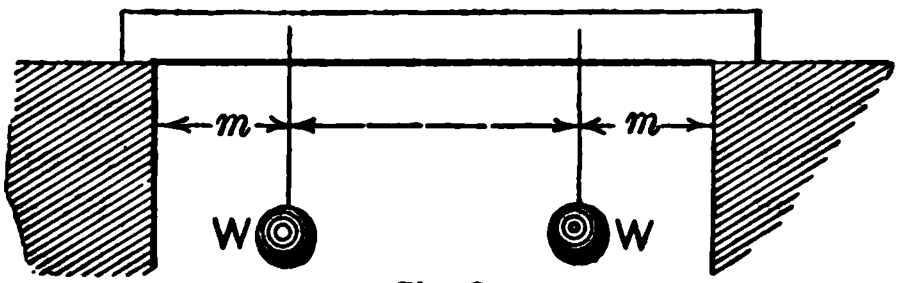


Fig. 6.

Safe load *W*, in pounds at each point =

$$\frac{1000 \times \text{moment of inertia}}{m \text{ in feet} \times y}. \quad (6)$$

The letter y in the above formulas is used to denote the distance of the farthest fibre from the neutral axis; and, in beams of symmetrical section, y would be one-half the height of the beam in inches. These formulas apply to iron beams of any form of cross-section, from an I-beam to an angle or T bar. For steel beams, increase the value of *W* one-third.

Weight of Beam to be subtracted from its Safe Load.

As the weight of iron beams often amounts to a considerable proportion of the load which they can carry, the weight should always be subtracted from the maximum safe load: for beams with concentrated loads, and for beams with distributed loads, one-half the weight of the beam should be subtracted.

EXAMPLE 1. — What is the safe load for a Trenton $12\frac{1}{4}$-inch light I-beam, 125 pounds per yard, having a clear span of 20 feet, the load being concentrated at a point 5 feet from one end?

Ans. Safe load (For. 5) $= \dfrac{1000 \times \text{I} \times \text{span}}{m \times n \times y} = \dfrac{1000 \times 288 \times 20}{5 \times 15 \times 6\frac{1}{8}} =$ **12,500 pounds.**

EXAMPLE 2.—A 12-inch Carnegie iron channel-bar, weighing 90 pounds per yard, and having a clear span of 24 feet, supports a concentrated load at two points, 6 feet from each end. What is the maximum load that can be supported at each point consistent with safety?

Ans. Safe load at each point $= \frac{1000 \times 173.7}{6 \times 6} = 4825$ pounds.

The moment of inertia for channels and angle-bars, and other sections, will be found in Chap. XIII.

Deepest Beam always most Economical.

Whenever we have a large load to carry with a given span, it will be found that it can be carried with the least amount of iron by using the deepest beams, provided the beams are not too strong for the load. Thus, suppose we wish to support a load of 9 tons with a span of 20 feet, by means of Trenton beams. We could do this either by one 12¼-inch beam at 125 pounds per yard, or by two 9-inch beams at 85 pounds per yard. But the 12¼-inch beam, 21 feet long, would weigh only 875 pounds, while the two 9-inch beams would weigh 1190 pounds; so that, by using the deeper beam, we save 315 pounds of iron, worth from three to five cents per pound.

The following table, under the heading $\frac{C}{W}$, gives the relative strength of Trenton beams in proportion to their weight, thus exhibiting the greater economy of the deeper patterns.

Trenton Rolled I-Beams.

STRENGTH OF EACH BEAM IN PROPORTION TO ITS WEIGHT.

BEAM.	$\frac{C}{W}$	BEAM.	$\frac{C}{W}$
15 inch, heavy	37.41	8 inch, light	20.75
15 " light	36.76	7 " 55 pounds . . .	18.37
12¼ " heavy	28.41	6 " 120 " . . .	14.33
12¼ " light	30.64	6 " 90 " . . .	14.67
10½ " heavy	26.64	6 " heavy	15.36
10½ " light	27.20	6 " light	15.65
10½ " extra light . . .	27.78	5 " heavy	12.27
9 " extra heavy . .	21.44	5 " light	12.90
9 " heavy	23.41	4 " heavy	9.95
" light	23.86	4 " light	10.03
" heavy	20.99	4 " extra light . . .	10.00

Another important advantage in the use of deeper beams is their greater stiffness. By referring to the tables, it will be seen that a beam twenty feet long, under its safe load, if 6 inches deep will deflect 0.95 inch; 9 inches deep, will deflect 0.63 inch; 12¼ inches deep, will deflect 0.46 inch; and 15 inches deep, will deflect only 0.38 inch.

A floor or structure formed of deep beams will therefore be much more rigid than one of the same strength formed of smaller sections.

There are, of course, cases where the use of deep beams would be inconvenient, either from increasing the depth of the floor, or from the fact that, with a light load and short span, they would have to be placed too far apart for convenience. In general, however, it will be best to employ the deep beams.

Inclined Beams.—The strength of beams inclined to the horizon may be computed, with sufficient accuracy for most purposes, by using the formulas given for horizontal beams, taking the horizontal projection of the beam as its span.

Steel and Iron Beams.—The relative efficiencies of steel and iron beams depend upon the conditions under which they are used. The transverse strength of beams of the same length and section is proportional to the tensile strength of the material, or beams made of steel, of 65,000 pounds tenacity, will possess an ultimate strength about 30 per cent. greater than similar beams made of iron of 50,000 pounds tenacity. But the steel beam will not be stiffer than the iron beam—that is, it will deflect under working loads as much as the iron beam of the same length and section; the steel beam merely bending farther than the iron beam without injury to its elasticity. Therefore, if strength without regard to stiffness is sought, the steel beam is the best; but if stiffness without regard to ultimate strength is desired, beams of either material would probably prove of equal utility.

Steel beams should not be used for their full load when the span in feet exceeds twice the depth of the beam in inches.

NOTE.—Since 1893 the Carnegie Steel Company has discontinued the manufacture of iron beams and bars for structural work, and now manufacture all their shapes in steel only. As steel beams, angles, etc., are sold at the same price per pound, and are about 20 per cent. stronger than iron, steel has naturally almost entirely superseded iron in rolled sections.

Strength of Trenton, Pencoyd, Phœnix, and Carnegie Rolled Beams, Channels, Angle and T-Bars—Iron and Steel.

The following tables give the strength and weight of the various sections to be found in the market, together with the general dimensions of the I-beams.

The tables are in all cases made up from data published by the

respective manufacturers. The deflection of the beams under their maximum safe distributed load is also given in some of the tables.

The tables on pages 349 to 363 will be found very convenient, for they can be used for the spans indicated, without any computations whatever. In these tables, the loads to the right of and below the heavy line will crack plastered ceilings. When 12- to 24-inch beams are used to their full capacity for spans less than 10 feet, the web should be stiffened at the ends.

RENGTH, WEIGHT, AND DIMENSIONS OF TRENTON ROLLED I-BEAMS—IRON.

	I.	II.	III.	IV.	V.
signation of beam.	Weight per yard, in lbs.	Safe distributed load for one foot of span, in lbs.*	Moment of inertia. Neutral axis perpendicular to web.	Width of flange, in ins.	Area of cross-section, in ins.
ch, heavy	272	1,320,000	....	6.75	27.20
light	200	990,000	....	6.00	20.00
heavy	200	748,000	707.1	5.75	20.02
light	150	551,000	523.5	5.00	15.04
light	125	460,000	434.5	5.00	12.36
heavy	170	511,000	391.2	5.50	16.77
light	125	377,000	288.0	4.79	12.33
heavy	120	375,000	281.3	5.50	11.73
light	96	306,000	229.2	5.25	9.46
heavy	135	360,000	233.7	5.00	13.36
light	105	286,000	185.6	4.50	10.44
extra light	90	250,000	164.0	4.50	8.90
extra heavy	125	268,000	150.8	4.50	12.33
heavy	85	199,000	111.9	4.50	8.50
light	70	167,000	93.9	4.00	7.00
heavy	80	168,000	83.9	4.50	8.03
light	65	135,000	67.4	4.00	6.37
55 lbs.	55	101,000	44.3	3.75	5.50
120 "	120	172,000	64.9	5.25	11.84
90 "	90	132,000	49.8	5.00	8.70
heavy	50	76,800	29.0	3.50	4.91
light	40	62,600	23.5	3.00	4.01
heavy	40	49,100	15.4	3.00	3.90
light	30	38,700	12.1	2.75	2.99
heavy	37	36,800	9.2	3.00	3.66
light	30	30,100	7.5	2.75	2.91
extra light.......	18	18,000	4.5	2.00	1.77

* For any other span divide this coefficient by span in feet.

STRENGTH, WEIGHT, AND DIMENSIONS OF TRENTON ROLLED I-BEAMS—STEEL.

	I.	II.	III.	IV.	V.
Designation of beam, in inches.	Weight per yard, in lbs.	Safe distributed load for one foot of span in, lbs. Fibre strain of 16,000 lbs.*	Moment of inertia. Neutral axis perpendicular to web.	Width of flange, in inches.	Area of cross-section, in inches.
15	150	753,000	529.7	5.75	14.70
15	123	603,000	424.4	5.5	12.02
12	120	500,000	281.3	5.5	11.73
12	96	407,000	229.2	5.25	9.46
10	135	461,000	216.1	5.25	13.14
10	99	344,000	161.3	5.0	9.67
10	76	264,000	123.6	4.75	7.50
9	81	262,000	110.6	4.75	7.98
9	63	200,000	84.3	4.5	6.15
8	66	192,000	71.9	4.5	6.47
8	54	154,000	57.7	4.25	5.28
7	60	151,000	49.7	4.25	5.87
7	46.5	118,000	38.6	4.0	4.55
6	50	104,000	29.2	3.5	4.97
6	40	83,300	23.4	3.0	3.97
5	39	67,000	15.7	3.13	3.80
5	30	52,900	12.4	3.0	2.96
4	30	41,200	7.7	2.75	2.94
4	22.5	31,400	5.9	2.63	2.21

* For any other span divide this coefficient by span.

ŁENGTH, WEIGHT, AND DIMENSIONS OF TRENTON CHANNEL-BARS AND DECK-BEAMS—IRON.

	I.	II.	III.	IV.	V.
esignation of bar.	Weight per yard, in lbs.	Safe distributed load, in lbs., for one foot of span.*	Moment of inertia I.	Width of flange, in ins.	Area of cross-section, in ins.
CHANNEL-BARS.					
ch, heavy	190	625,000	586.0	4¾	18.85
' light	120	401,000	376.0	4	12.00
' heavy	140	381,000	291.6	4	14.10
' light	70	200,100	153.2	3	7.00
' light	60	134,750	88.4	2¾	6.00
' heavy	48	102,500	64.0	2½	4.77
' heavy	70	146,000	82.1	3⅛	7.02
' light	50	104,000	58.8	2¼	5.08
' light	45	88,950	44.5	2¼	4.48
' extra light.......	33	65,800	32.9	2.2	3.30
' light	36	62,000	27.1	2½	3.60
' extra light	25¼	39,500	17.3	2	2.54
' heavy	45	58,300	21.7	2½	4.32
' light	33	45,700	17.2	2¼	3.20
' extra light	22¼	33,680	12.6	1⅞	2.25
' extra light	19	22,800	7.2	1⅝	1.92
' extra light	16½	15,700	3.9	1½	1.65
' extra light	15	10,500	2.0	1½	1.45
DECK-BEAMS.					
ch	65	91,800	54.7	4½	6.29
'	55	63,500	35.1	4½	5.35

* For coefficient of steel bars add one-third.

STRENGTH, WEIGHT, AND DIMENSIONS OF TRENTON ANGLE AND T BARS.

	I.	II.		I.	II.
Designation of bar.	Weight per foot, in lbs.	Safe distributed load for one foot of span, in lbs.	Designation of bar.	Weight per foot, in lbs.	Safe distributed load for one foot of span, in lbs.
ANGLES EVEN LEGS.			**ANGLES UNEQUAL LEGS.**		
6 in. × 6 in.	19.00	36,900	6 in. × 4 in.	14.00	30,680
4½ " × 4½ "	12½	18,000			14,750
4 " × 4 "	9¼	12,184	5 " × 3½ "	10.20	18,353
3½ " × 3½ "	8¼	9,200			9,651
3 " × 3 "	4.80	4,611	4½ " × 3 "	9.00	14,580
2¾ " × 2¾ "	5.40	4,710			7,020
2½ " × 2½ "	3.90	3,156	4 " × 3 "	7.00	9,850
2¼ " × 2¼ "	3.50	2,530			5,871
2 " × 2 "	3.13	1,970	3½ " × 1½ "	4.00	5,515
1¾ " × 1¾ "	2.00	1,150			1,148
1½ " × 1½ "	1.75	832	3 " × 2½ "	4.37	4,490
1¼ " × 1¼ "	1.00	393			3,233
1 " × 1 "	0.75	246	3 " × 2 "	4.00	4,334
⅞ " × ⅞ "	0.60	186			2,080
¾ " × ¾ "	0.56	133			
T-BARS.					
4 in. × 4 in.	12.50	15,800	3 in. × 2 in.	4.80	2,540
3½ " × 3½ "	9.60	10,550	2 " × 1½ "	3.00	1,355
3 " × 3 "	7.00	6,680	2¼ " × 1¼ "	2.40	604
2½ " × 2½ "	5.00	3,850	2 " × 1 "	2.15	457
2 " × 2 "	3.13	1,970	1½ " × 1 "	1.86	421
5 " × 2½ "	11.70	6,344			

* For coefficient of steel bars add one-third. For any other span divide this coefficient by span.

STRENGTH, WEIGHT, AND DIMENSIONS OF CARNEGIE I-BEAMS—STEEL.

Depth of beam, in inches.	Weight per foot, in lbs.	Thickness of web, in inches.	Width of flange. in inches.	Safe distributed load for one foot of span, in lbs. 16,000 lbs. fibre strain for buildings.*	Safe distributed load for one foot of span, in lbs. 12,500 lbs. fibre strain for bridges.*
24	100	.75	7.20	2,086,500	1,670,000
24	80	.50	6.95	1,830,500	1,466,000
20	80	.60	7.00	1,545,600	1,207,500
20	64	.50	6.25	1,222,400	955,000
15	75	.67	6.31	1,077,300	841,700
15	60	.54	6.04	916,300	715,800
15	50	.45	5.75	753,300	588,500
15	41	.40	5.50	603,200	471,300
12	40	.39	5.50	500,100	390,700
12	32	.35	5.25	395,200	308,800
10	33	.37	5.00	344,000	268,800
10	25.5	.32	4.75	263,800	206,100
9	27	.31	4.75	262,200	204,900
9	21	.27	4.50	199,900	156,100
8	22	.27	4.50	191,600	149,700
8	18	.25	4.25	154,000	120,300
7	20	.27	4.25	151,400	118,300
7	15.5	.23	4.00	117,600	91,900
6	16	.26	3.63	101,800	79,500
6	13	.23	3.50	83,500	65,300
5	13	.26	3.13	67,000	52,400
5	10	.22	3.00	52,900	41,300
4	10	.24	2.75	41,200	32,200
4	7.5	.20	2.63	31,400	24,600

* For any other span divide this coefficient by span.

STRENGTH, WEIGHT, AND DIMENSIONS OF CARNEGIE CHANNEL-BARS—IRON.

* For any other span divide this coefficient by span.

STRENGTH, WEIGHT, AND DIMENSIONS OF CARNEGIE CHANNEL-BARS—STEEL.

Depth of channel, in inches.	Weight per foot, in lbs.	Thickness of web, in inches.	Width of flange, in inches.	Safe distributed load for one foot of span, in lbs. 16,000 lbs. fibre strain for buildings.*	Safe distributed load for one foot of span, in lbs. 12,500 lbs. fibre strain for bridges.*
15	32	.40	3.40	404,700	316,200
15	51	.775	3.775	554,700	433,400
12	20	.30	2.90	209,600	163,800
12	30¼	.55	3.15	273,600	213,800
10	15¼	.26	2.66	136,100	106,300
10	23¾	.51	2.91	180,500	141,000
9	12¾	.24	2.44	102,700	80,200
9	20½	.49	2.69	138,700	108,400
8	10½	.22	2.22	75,300	58,800
8	17½	.47	2.47	103,700	81,000
7	8½	.20	2.00	53,100	41,500
7	14½	.45	2.25	75,000	58,600
6	7	.19	1.85	39,400	30,800
6	12	.44	2.14	55,400	43,300
5	6	.18	1.78	27,900	21,800
5	10¼	.43	2.03	39,000	30,500
4	5	.17	1.67	18,700	14,600
4	8¼	.42	1.92	25,700	20,100

* For any other span divide this coefficient by span.

STRENGTH, WEIGHT, AND DIMENSIONS OF JONES & LAUGHLIN'S, LIMITED, STEEL BEAMS.

Depth of beam, in inches.	Weight per foot, in lbs.	Thickness of web, in inches.	Width of flange, in inches.	Safe distributed load for one foot of span, in lbs. 16,000 lbs. fibre strain for buildings.*	Safe distributed load for one foot of span, in lbs. 12,000 lbs. fibre strain for bridges.*
15	70	0.64	6.265	1,039,700	810,700
15	59	0.468	5 968	910,000	710.900
15	48	0.406	5.726	705,200	550,900
15	39	0.375	5.475	573,500	448,000
12	50	0.598	5.723	536,800	419,400
12	38	0.343	5.468	471,800	368,600
12	30	0.312	5.218	376,400	294,100
10	32	0.3125	4.937	325,500	254,300
10	23.8	0.281	4.72	251,100	196,200
9	24.5	0.296	4.671	239,700	187,300
9	19.75	0.266	4.39	189,100	147,700
8	25	0.287	4.537	191,500	149,600
8	18	0.25	4.25	152,800	119.400
7	18.3	0.266	4.266	141,400	110,500
7	15.25	0.25	4.0	115,500	90,200
6	16.6	0.265	3.765	100,900	78,800
6	12.75	0.25	3.5	82,100	64,100
5	13	0.31	3.06	67,000	52,300
5	10	0.22	2.845	57,600	45,000
4	10.2	0.28	2.78	41,100	32,100
4	7.9	0.25	2.69	32,000	25,000
4	6.85	0.19	2.56	31,000	24,200
3	7	0.19	2.32	22,000	17,200
3	5.1	0.156	2.03	16,300	12,700

* For any other span divide this coefficient by span.

STRENGTH, WEIGHT, AND DIMENSIONS OF PHŒNIX I-BEAMS—STEEL.

Depth of beam, in inches.	Weight per yard, in lbs.	Thickness of web, in inches.	Width of flange, in inches.	Safe distributed load for one foot of span, in lbs. 16,000 lbs. fibre strain for buildings.*	Safe distributed load for one foot of span, in lbs. 12,500 lbs. fibre strain for bridges.*
15	225	.62	6.375	1,076,000	840,600
15	180	.50	6.125	920,000	718,750
15	150	.45	5.75	752,000	587,500
15	123	.40	5.50	602,000	470,300
12	120	.39	5.50	500,000	390,600
12	96	.35	5.25	394,000	307,800
10½	99	.35	5.00	368,000	287,500
10½	76½	.30	4.75	284,000	221,800
9	81	.31	4.75	262,000	204,600
9	63	.27	4.50	200,000	156,200
8	66	.27	4.50	190,000	148,400
8	54	.25	4.25	154,000	120,300
7	60	.27	4.25	142,000	110,900
7	46½	.23	4.00	114,000	89,060
6	48	.26	3.625	100,000	78,120
6	39	.23	3.50	82,000	64,060
5	39	.26	3.125	66,000	51,560
5	30	.22	3.00	52,000	40,620
4	30	.24	2.75	40,000	31,250

* For any other span divide this coefficient by span.

Pencoyd Beams and Channels.

The coefficient for strength of the Pencoyd sections has been calculated for a fibre strain of 14,000 lbs. for iron, and 16,500 lbs. for steel.

These tables also contain the maximum load that should be placed on the beam, *whatever the length*, unless the web is stiffened at the points of support.

EXAMPLE.—What should be the maximum distributed load for a 15-inch 145-lb. iron beam of 10 feet span? *Ans.* The coefficient of this beam is 648,600 lbs. Dividing by 10, we have 64,860 lbs., or 32.4 tons as the safe load; but we see, by the last column, that it will not be safe to put more than 22.1 tons on the beam without stiffening the web. Hence, the safe load for that span is 22.1 tons. It is only for very short beams that this condition will apply.

STRENGTH, WEIGHT, AND DIMENSIONS OF PENCOYD I-BEAMS—STEEL.

Depth of beam, in inches.	Weight per yard, in lbs.	Thickness of web, in inches.	Width of flange, in inches.	Safe distributed load for one foot of span, in lbs. 14,000 lbs. fibre strain for buildings.*	Maximum load in tons, without stiffening web.
10	70.1	.30	4.50	242,260	12.06
9	60.1	.28	4.30	193,040	10.44
8	51.7	.26	4.00	146,360	8.98
7	43.4	.24	3.75	108,840	7.50
6	34.9	.22	3.40	76,160	6.18
5	27.3	.20	3.00	49,900	4.94
4	25.0	.22	2.6	35,360	5.05
4	18.6	.16	2.3	27,120	3.16
3	20.5	.22	2.4	21,480	3.77
3	15.9	.16	2.2	17,380	2.72

* For any other span divide this coefficient by span. The load, however, must be greater than that in next column, unless the web is stiffened at supports

STRENGTH, WEIGHT, AND DIMENSIONS OF PENCOYD I-BEAMS—IRON.

Depth of beam, in inches.	Weight per yard, in lbs.	Thickness of web, in inches.	Width of flange, in inches.	Safe distributed load for one foot of span, in lbs. 14,000 lbs. fibre strain for buildings.	Maximum load in tons, without stiffening web.
15	190.0	.562	5.687	844,560	39.57
15	145.0	.437	5.125	648,600	22.10
15	124.1	.406	5.609	541,980	18.59
12	168.0	.656	5.5	578,640	38.63
12	120.0	.453	4.80	424,440	22.22
12	89.5	.343	5.0	317,440	13.60
10½	134.4	.468	5.25	429,560	22.13
10½	108.3	.406	4.87	347,420	17.71
10½	89.3	.343	4.5	288,460	13.35
10	111.7	.5	4.625	324,040	23.68
10	90.4	.343	4.375	276,860	13 18
9	90.0	.406	4.75	246,420	16.53
9	70.6	.312	4.25	195,880	9.94
8	80.0	.406	4.375	188.840	13.83
8	61.0	.297	4.0	161,400	10.46
7	65.8	.437	3.20	132,760	15.69
7	51.4	.234	3.61	114,880	6.17
6	115.5	.625	5.25	196,740	21.19
6	90.1	.5	4.87	160,000	16.42
6	55.5	.281	3.84	103,480	7.75
6	40.0	.218	3.47	76,500	5.25
5	29.7	.26	3.0	46,560	4.91
4	24.6	.22	2.6	30,000	4.33
4	18.2	.16	2.3	23,000	2.71
3	20.1	.22	2.4	19,240	3.23
3	15.6	.16	2.2	14,740	2.33

SAFE DISTRIBUTED LOADS AND DEFLECTIONS OF PENCOYD CHANNELS—IRON.

Greatest safe load in net tons, evenly distributed, including beam itself. For a concentrated load in middle of beam allow one-half of that given in table below.

Size, in ins.	Weight per yard, in lbs.	5	6	8	10	12	14	16	18	20	22	24
15	130.0				27.00	22.50	19.28	16.87	15.00	13.50	12.27	11.25
15	100.0				21.88	18.23	15.63	13.67	12.15	10.94	9.94	9.12
12	88.5				14.21	11.84	10.15	8.88	7.90	7.10	6.46	5.92
12	60.0				9.62	8.02	6.87	6.01	5.34	4.81	4.37	4.01
12	[illegible]				10.32	8.60	7.37	6.45	5.73	5.16	4.69	4.30
10	[illegible]				8.05	[illegible]						
10	[illegible]				[illegible]	[illegible]						
10	[illegible]		10.11	7.58	6.06	[illegible]						
9	[illegible]		[illegible]	5.67	4.54	[illegible]						
8	[illegible]		7.77	5.83	4.66	[illegible]						
8	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]						
8	[illegible]		[illegible]	4.25	3.40	[illegible]						
7	[illegible]	7.86	6.55	4.91	3.93	[illegible]						
7	[illegible]	[illegible]	4.12	[illegible]	[illegible]	[illegible]						
6	[illegible]	5.72	4.76	3.57	2.86	[illegible]						
6	[illegible]	4.50	3.75	[illegible]	2.25	1.[illegible]						
6	[illegible]	3.62	3.02	2.26	1.81	1.[illegible]						
5	[illegible]	3.84	3.20	2.40	1.[illegible]							
5	[illegible]	2.48	2.07	1.55	1.[illegible]							
4	[illegible]	2.42	2.02	1.[illegible]								
4	[illegible]	1.92	1.60	1.[illegible]								
[illegible]	[illegible]	1.26	1.05									

[illegible] figures represent [illegible]. For deflect[illegible] of the tab[illegible] can be [illegible] The safe loa[illegible] equal load[illegible]

SAFE DISTRIBUTED LOADS OF PHŒNIX BEAMS—STEEL.

Greatest safe load in net tons, evenly distributed. For a concentrated load in middle of beam, allow one-half of that given in the table.

Size of beam, in inches.	Weight per yard, in lbs.	Clear span, in feet.									
		10	12	14	16	18	20	22	24	26	28
15	225	53.90	44.90	38.48	33.67	29.93	26.93	24.48	22.44	20.72	19.24
15	190	45.82	38.18	32.72	28.63	25.45	22.91	20.83	19.09	17.62	16.36
15	150	37.66	31.39	26.90	23.54	20.93	18.83	17.12	15.69	14.48	13.45
15	125	30.17	25.13	21.54	18.85	16.75	15.08	13.71	12.57	11.60	10.77

STRENGTH, WEIGHT, AND DIMENSIONS OF PENCOY CHANNELS.

For Steel.

Depth of channel, in inches.	Weight per yard, in lbs.	Thickness of web, in inches.	Width of flange, in inches.	Safe distributed load for one foot of span, in lbs. 14,000 lbs. fibre strain for buildings.	Maximum load in tons without stiffening web.
8	31.3	.22	2.27	79,020	6.55
7	26.6	.21	2.11	79,080	5.91
6	22.2	.20	1.95	42,600	5.25
5	18.1	.19	1.79	29,260	4.55
4	14.7	.18	1.63	19,300	3.79

For Iron.

Depth of channel, in inches.	Weight per yard, in lbs.	Thickness of web, in inches.	Width of flange, in inches.	Safe distributed load for one foot of span, in lbs. 14,000 lbs. fibre strain for buildings.	Maximum load in tons without stiffening web.
15	139.0	.562	3.94	539,940	34.34
15	106.0	.375	3.87	437,500	16.33
12	88.5	.406	2.94	284,220	18.49
12	60.0	.281	2.61	192,440	9.14
12	61.5	.281	3.09	206,460	9.06
10	59.7	.328	2.75	164,740	13.57
10	47.5	.25	2.5	133,660	8.46
9	52.7	.312	2.59	125,740	13.20
9	37.2	.234	2.36	92,540	7.17
8	43.0	.281	2.23	93,320	8.77
8	39.5	.25	2.50	80,800	7.65
8	30.7	.218	2.28	58,240	4.65
7	41.0	.297	2.30	78,700	9.07
7	25.0	.171	1.95	49,220	3.42
6	31.9	.25	2.25	57,160	6.50
6	22.7	.20	1.75	36,320	5.94
5	23.9	.23	2.06	34,120	5.24
4	21.5	.25	1.69	24,060	5.12
4	16.5	.19	1.26	19,300	4.29
3	15.2	.22	1.53	12,640	3.49
2½	11.3	.25	1.37	6,660	3.20
[illegible]	8.8	.22	1.09	4,500	2.49

SAFE DISTRIBUTED LOADS AND DEFLECTIONS OF PENCOYD BEAMS—IRON.

Greatest safe load in net ... concentrated

For a ... below.

Size of beam, in ins.
15
15
15
12
12
12
10½
10½
10½
10 *
10 *
9 *
9 *
8 *
8 *
7 *
7 *
6 *
6 *
6 *
6 *
5 *
4 *
4 *
3 *
3 *

The figures in italics represent the deflections in inches corresponding to the safe loads above. For deflections corresponding to greatest safe load in middle, take four fifths (⅘) of the tabular figures.

Beams marked * can be rolled in steel, when the weights will be increased 2 per cent.—safe load about 20 per cent. Deflection practically the same as for iron with equal loads.

SAFE DISTRIBUTED LOADS AND DEFLECTIONS OF TRENTON STEEL BEAMS.

Safe loads in net tons evenly distributed (in addition to weight of beam). For a concentrated load in middle, allow one-half of that given in table below.

NOTE.—The figures in italics are the deflections, in inches, corresponding to loads above. For the deflections of greatest safe loads in middle, take four (⅘) of the tabular figures in italics.

SAFE DISTRIBUTED LOADS OF CARNEGIE IRON BEAMS.

Safe loads in net tons evenly distributed (including weight of beam). For a concentrated load in middle, allow one-half of that given in table below.

Size of beam, in inches.	Weight per yard, in lbs.	Length of span, in feet.
15	240	
15	200	
15	150	
12	168½	
12	125	
10½	135	
10½	94½	
10	126	
9	108	
9	90	
9	115½	
[illegible]	85½	
[illegible]	70½	
[illegible]	108	
8	81	
8	64½	
7	66	
7	54	
6	48	
6	40½	
5	36	
5	30	
4	21	
4	18	
3	27	
3	16½	

NOTE.—Loads to the right and below heavy line will crack plastered ceilings.

SAFE DISTRIBUTED LOADS, IN TONS OF 2,000 LBS., FOR CARNEGIE STEEL BEAMS.

For fibre strains of $10\frac{2}{3}$, 8, and 6 tons. Factor of safety of 3, 4, and 5.

Explanation. Beams having loads and spans to the *left* of the black line will not crack plastered ceilings.

Beams having loads and spans to the *right* of *dotted* line should have their upper flanges secured from buckling, due to the compressive strain in same.

Deflections in 64ths of an inch for a 10-foot span are given. For any other span and the corresponding safe load, multiply this by the square of the span in feet and divide by 100.

For beams supported at both ends and loaded at the centre, take one-half tabular load.

For beams fixed at one end and loaded at the other, take one-eighth tabular load.

For beams fixed at one end and uniformly loaded, take one-fourth tabular load.

Deflections are, respectively, 8, 3.2, and 2.4 of that in tables.

15	75	3
		4
		5
15	60	3
		4
		5
15	50	3
		4
		5
15	41	3
		4
		5
12	40	3
		4
		5
12	32	3
		4
		5
10	35	3
		4
		5
10	25.5	3
		4
		5
9	27	3
		4
		5
9	21	3
		4
		5

SAFE LOADS OF T-

Greatest weight dis
load allo

SAFE DISTRIBUTED LOADS FOR CARNEGIE ANGLE-IRONS.

Equal Legs.

Greatest safe load in lbs. uniformly distributed, including weight of angle-iron, for 12,000 lbs. fibre strain

For concentrated load in middle of beam allow one-half of that given in the table.

Size of angle, in inches.
6 × 6 × 7/16
6 × 6 × 3/8
5 × 5 × 5/8
5 × 5 × 7/8
4 × 4 × 5/8
4 × 4 × 13/16
3½ × 3½ × 3/8
3½ × 3½ × 11/16
3 × 3 × 3/8
3 × 3 × 5/8
2¾ × 2¾ × ½
2¾ × 2¾ × ¼
2½ × 2½ × ½
2½ × 2½ × ¼
2¼ × 2¼ × ½
2¼ × 2¼ × ¼
2 × 2 × 5/16
2 × 2 × 7/16
1¾ × 1¾ × 5/16
1½ × 1½ × 7/16
1½ × 1½ × 5/16
1¼ × 1¼ × 5/8
1¼ × 1¼ × ¼
1¼ × 1¼ × ½

SAFE DISTRIBUTED LOADS FOR CARNEGIE ANGLE-IRONS.

Angles with Unequal Legs—Long Leg Vertical.

Greatest safe load in lbs. uniformly distributed, including weight of angle-iron, for 12,000 lbs. fibre strain. For concentrated load in middle of beam allow one-half of that given in the table.

5 × 3 × 3/8	9.6
5 × 3 × 13/16	20.1
4½ × 3 × 3/8	8.9
4½ × 3 × 13/16	18.7
4 × 3½ × 3/8	8.9
4 × 3½ × 13/16	16.7
4 × 3 × 5/16	7.0
4 × 3 × 13/16	17.4
3½ × 3 × 5/16	6.5
3½ × 3 × 13/16	16.0
3½ × 2½ × ¼	4.8
3½ × 2½ × ½	9.8
3 × 2½ × ¼	4.4
3 × 2½ × ½	8.7
3 × 2 × ¼	4.0
3 × 2 × ½	8.0
2½ × 2 × 11/16	2.7
½ × 2 × ½	7.2

SAFE DISTRIBUTED LOADS FOR CARNEGIE ANGLE-IRONS.*

Angles with Unequal Legs—Short Leg Vertical.

Greatest safe load in lbs. uniformly distributed, including weight of angle-iron, for 12,000 l concentrated load in middle of beam allow one-half of that

Beams Supporting Brick Walls.

In the case of iron beams supporting brick walls having no openings, and in which the bricks are laid with the usual bond, the prism of wall that the beam sustains will be of a triangular shape, the height being one-fourth of the span. Owing to frequent irregularities in the bonding, it is best to consider the height as one-third of the span.

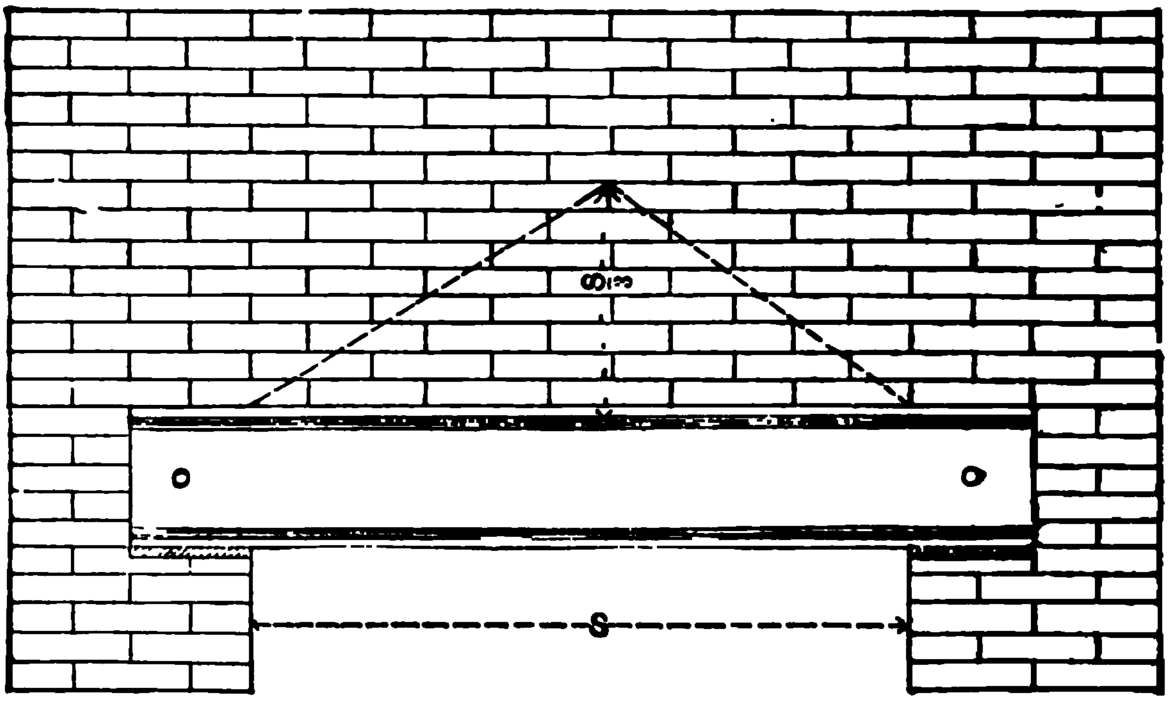


Fig. 7.

The greatest bending-stress at the centre of the beam, resulting from a brick wall of the above shape, is the same as that caused by a load one-sixth less, concentrated at the centre of the beam, or two-thirds more, evenly distributed.

The weight of brickwork is very nearly ten pounds per square foot for one inch in thickness; and from this data we find that the bending-stress on the beams would be the same as that caused by a uniformly distributed load equal to

$$\frac{25 \times \text{square of span in feet} \times \text{thickness in inches}}{9}$$

Having ascertained this load, we have merely to determine from the proper tables the size of beams required to carry a distributed load of this amount.

EXAMPLE. — It is proposed to support a solid brick wall 12 inches thick, over an opening 12 feet wide, on rolled iron beams: should be the size and weight of beams?

s. By the rule given above, the uniformly distributed load

which would produce the same bending-stress on the beam as the wall, equals

$$\frac{25 \times 144 \times 12}{9} = 4800 \text{ pounds.}$$

As the wall is twelve inches thick, it would be best to use two beams placed side by side to support it, as they would give a greater area to build the brick on ; then the load on each beam would be 2400 pounds, or 1.2 tons. From the preceding tables for safe distributed loads on beams, we find that a 4-inch heavy beam would just about support this load; but as a 5-inch light beam would not weigh any more, and would be much stiffer, it would be better for us to use two 5-inch light beams to support our wall.

If a wall has openings, such as windows, etc., the imposed weight on the beam may be greater than if the wall is solid.

For such a case consider the outline of the brick which the beam sustains to pass from the points of support diagonally to the outside corners of the nearest openings, then vertically up the outer line of the jambs, and so on, if other openings occur above. If there should be no other openings, consider the line of imposed brickwork to extend diagonally up from each upper corner of the jambs, the intersection forming a triangle whose height is one-third of its base, as described above.

When beams are used to support a wall entirely (that is, the beams run under the whole length of the wall), and the wall is more than sixteen or eighteen feet long, the whole weight of the wall should be taken as coming upon the beams; for, if the beams should bend, the wall would settle, and might push out the supports, and thus cause the whole structure to fall.

Framing and Connecting Iron Beams.

When beams are used to support walls, or as girders to carry floor-beams, they are often placed side by side, and should in such

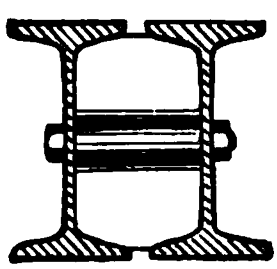


Fig. 8.

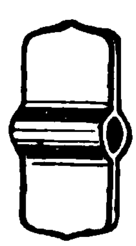


Fig. 9.

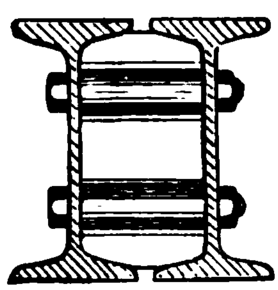


Fig. 10.

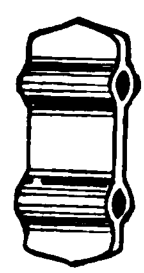


Fig. 11.

cases be furnished with cast-iron separators fitting between the flanges, so as to firmly combine the two beams. These separators may be placed from four to six feet apart. Such an arrangement is shown by Figs. 8 and 10, Figs. 9 and 11 showing forms of sepa-

rators usually employed; that with two bolt-holes being used the 15-inch and 12¼-inch beams, and that with a single hole smaller sizes.

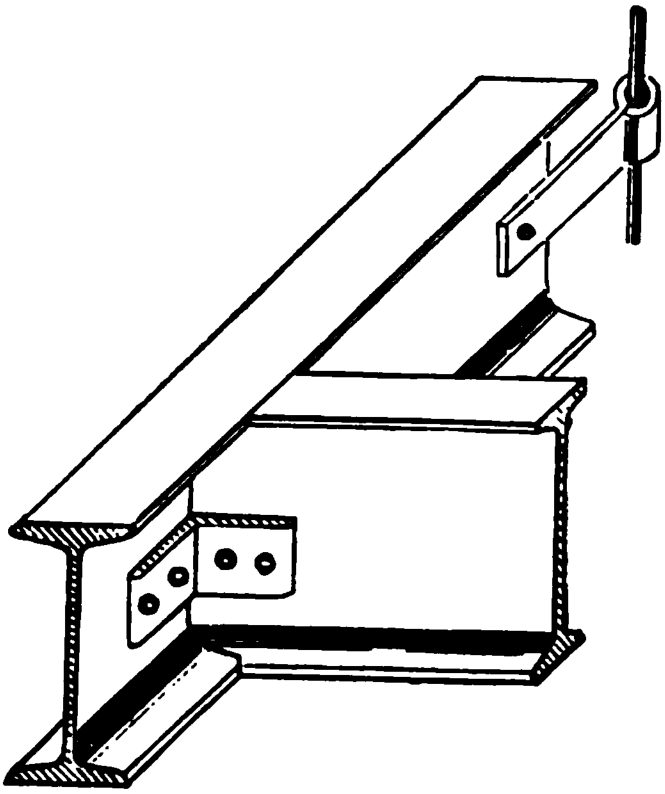


Fig. 12.

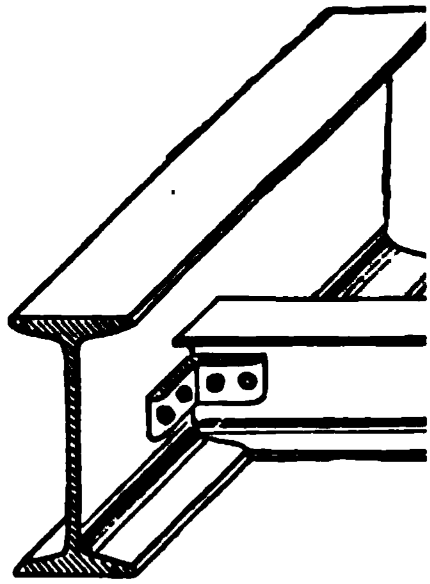


Fig. 13.

When beams are required to be framed together, it is usu done as shown by the accompanying cuts, in which Fig. 12 sh two beams of the same size fitted together. Fig. 13 shows a b fitted flush with the bottom flange of a beam of larger size. 14 shows a smaller beam fitted to the stem of a larger beam, al the lower flange.

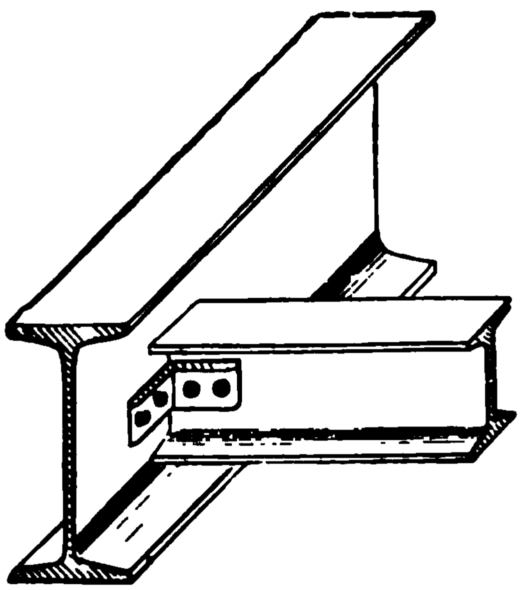


Fig. 14.

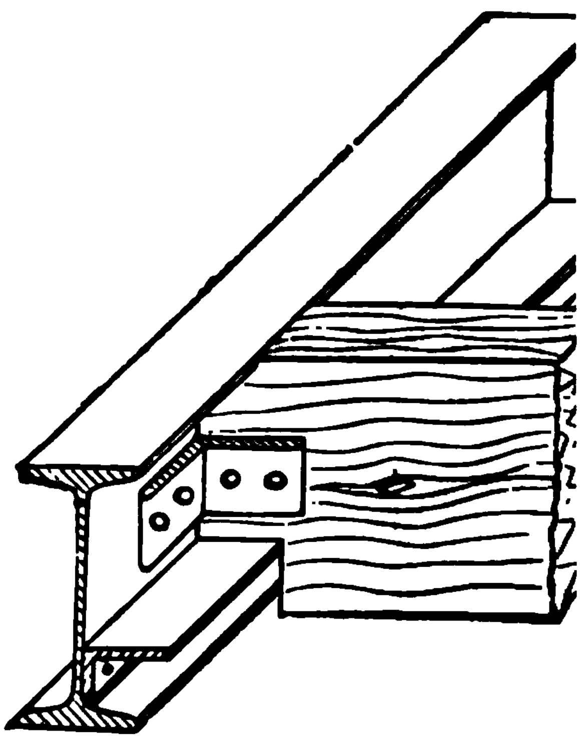


Fig. 15.

Wooden beams may be secured to an iron girder in the s manner as an iron beam, by framing the end, and securing it by -bracket; or an angle-iron may be riveted to the web of on girder to afford a flat bearing on which the wooden beam

The different rolling mills have standard connections for connecting iron beams with each other.

The standard connection angles for all sizes and weights of steel and iron I-beams manufactured by Carnegie, Phipps & Co., Limited, are illustrated on page 368. These connections were designed on the basis of an allowable shearing strain of 10,000 lbs. per square inch, and a bearing strain of 20,000 lbs. per square inch on rivets or bolts, corresponding with extreme fibre strains in the I-beams of 16,000 and 12,000 lbs. per square inch, for steel and iron respectively. The number of rivets or bolts required was found to be dependent, in most instances, on their bearing values.

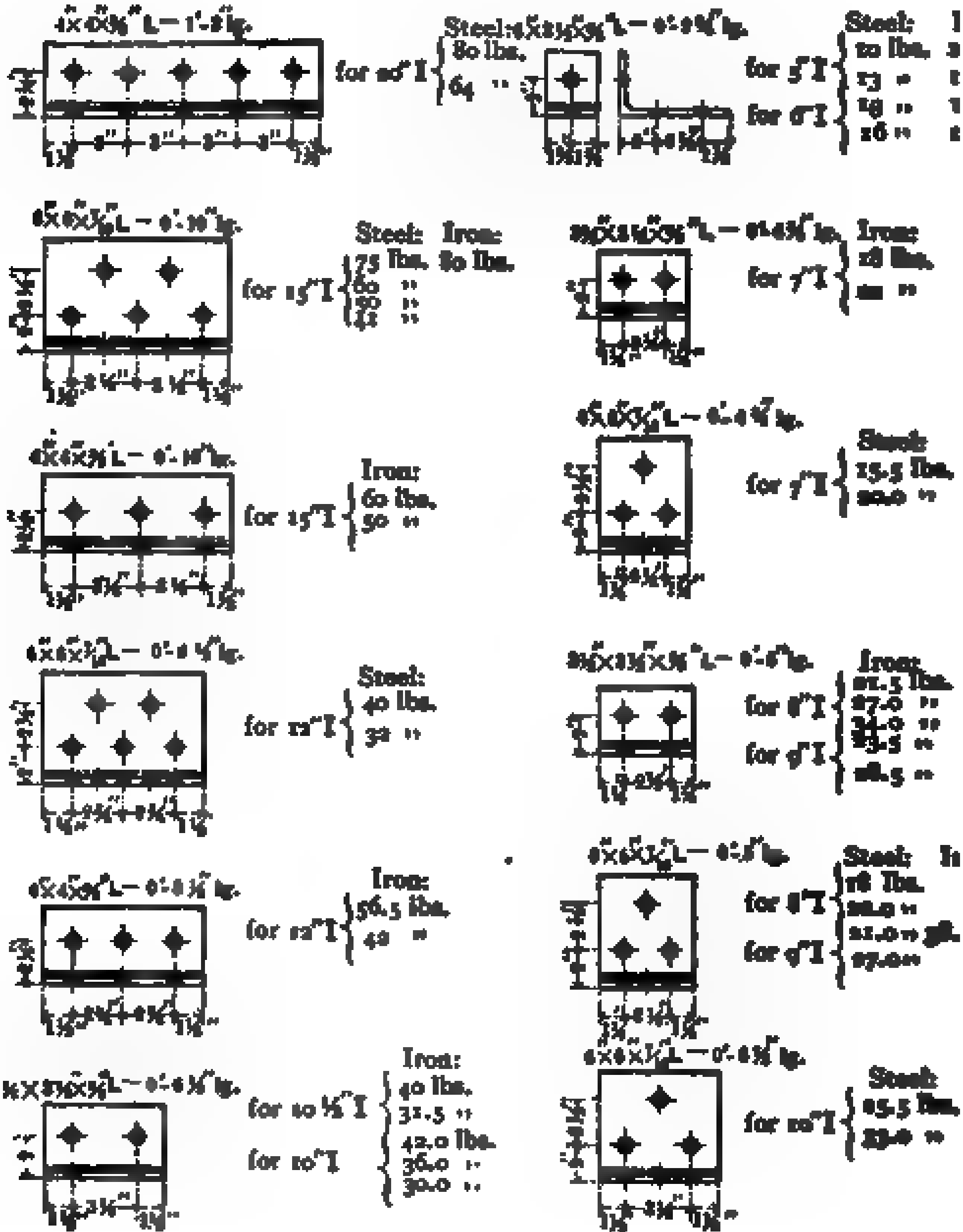
The connections have been proportioned with a view to covering most cases occurring in ordinary practice, with the usual relations of depth of beam to length of span. In extreme instances, however, where beams of short relative span lengths are loaded to their full capacity, it may be found necessary to make provision for additional strength in the connections. The limiting span lengths, at and above which the standard connection angles may be used with perfect safety, are given in the following table:

TABLE OF MINIMUM SPANS, FOR CARNEGIE I-BEAMS, WHERE STANDARD CONNECTION ANGLES MAY BE SAFELY USED, WITH BEAMS LOADED TO THEIR FULL CAPACITY.

Steel I-Beams.				Iron I-Beams.			
Designation of beam.	Minimum safe span, in feet.	Designation of beam.	Minimum safe span, in feet.	Designation of beam.	Minimum safe span, in feet.	Designation of beam.	Minimum safe span, in feet.
20″—80. lbs.	17.0	9″—27. lbs.	9.5	15″—80. lbs.	10.0	9″—28.5 lbs.	8.0
“ 64. “	16.0	“ 21. “	8.0	“ 60. “	13.0	“ 23.5 “	8.0
15″—75. “	12.0	8″—22. “	8.0	“ 50. “	13.0	8″—34. “	7.0
“ 60. “	11.5	“ 18. “	7.0	12″—56.5 “	9.0	“ 27. “	7.0
“ 50. “	11.0	7″—20. “	6.0	“ 42. “	8.0	“ 21.5 “	6.5
“ 41. “	10.5	“ 15.5 “	5.5	10½″—40. “	9.0	7″—22. “	5.0
12″—40. “	8.5	6″—16. “	6.5	“ 31.5 “	10.0	“ 18. “	6.5
“ 32. “	7.5	“ 13. “	6.0	10″—42. “	10.5	6″—16. “	5.0
10″ 33. “	10.5	5″—13. “	4.0	“ 36. “	10.5	“ 13.5 “	4.5
“ 25.5 “	9.0	“ 10. “	4.0	“ 30. “	10.5	5″—12. “	3.0
....	....	....	....	9″—38.5 “	6.5	“ 10. “	3.0

Fig. 2

STANDARD CONNECTION ANGLES FOR I BEAMS:

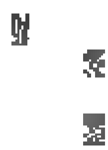


All holes for ¾" Bolts or Rivets.

SIZES AND WEIGHTS OF SEPARATORS FOR CARNEGIE STEEL BEAMS.

Separators for 20'' beams are made of $\frac{3}{8}$'' metal.
Separators for 8'' to 15'' beams are made of $\frac{5}{16}$'' metal
Separators for 5'' beams and under are made of $\frac{1}{4}$'' metal.

SEPARATORS WITH TWO BOLTS.



SEPARATORS WITH ONE BOLT.

SIZES AND WEIGHTS OF SEPARATORS FOR CARNEGIE IRON BEAMS.

Separators for 6″ beams and upward are made of ¼″ metal.
Separators for 5″ beams and under are made of ⅛″ metal.

SEPARATORS WITH TWO BOLTS.

SEPARATORS WITH ONE BOLT.

12	3b	56½	10⅞	5¾
12	3a	42	9⅞	5¼
10½	4b	40	10$\frac{1}{16}$	5¼
10½	4a	31½	9½	5
10	7	42	10	5¼
10	5b	36	9½	[illegible]
10	5a	30	9$\frac{1}{16}$	4¾
9	6c	38½	10	5¼
9	6b	28½	8⅝	4½
9	6a	23½	8½	4½
8	8c	34	9½	5
8	8b	27	8⅝	4½
8	8a	21½	8	4¼
7	9b	23	8$\frac{1}{16}$	4¼
7	9a	18	7½	4
6	10b	16	7$\frac{3}{16}$	4
6	10a	13½	7	3¾
5	11b	12	6½	3½
5	11a	10	6⅛	3¼
4	12	7	5½	3

CHAPTER XV.

STRENGTH OF CAST-IRON, WOODEN, AND STONE BEAMS — SOLID BUILT BEAMS.

Cast-Iron Beams. — Most of our knowledge of the strength of cast-iron beams is derived from the experiments of Mr. Eaton Hodgkinson. From these experiments he found that the form of cross-section of a beam which will resist the greatest transverse strain is that shown in Fig. 1, in which the bottom flange contains six times as much metal as the top flange.

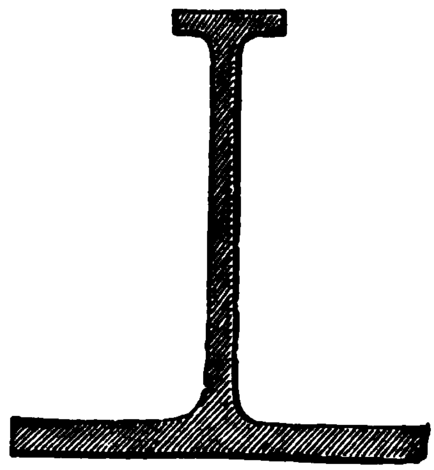


Fig 1.

When cast-iron beams are subjected to very light strains, the areas of the two flanges ought to be nearly equal. As in practice it is usual to submit beams to strains less than the ultimate load, and yet beyond a slight strain, it is found, that when the flanges are as 1 to 4, we have a proportion which approximates very nearly the requirements of practice. The thickness of the three parts — web, top flange, and bottom flange — may with advantage be made in proportion as 5, 6, and 8.

If made in this proportion, the width of the top flange will be equal to one-third of that of the bottom flange. As the result of his experiments, Mr. Hodgkinson gives the following rule for the breaking-weight at the centre for a cast-iron beam of the above form: —

$$\text{Breaking-load in tons} = \frac{\begin{matrix}\text{Area of bot. flange}\\ \text{in square inches}\end{matrix} \times \begin{matrix}\text{depth}\\ \text{in ins.}\end{matrix} \times 2.426}{\text{clear span in feet}}. \quad (1)$$

Cast-iron beams should always be tested by a load equal to that which they are designed to carry.

Wooden Beams. — Wooden beams are almost invariably square or rectangular shaped timbers, and we shall therefore consider only that shape in the following rules and formulas.

For beams with a rectangular cross-section, we can simplify our formulas for strength by substituting for the moment of inertia its value, viz., $\frac{b \times d^3}{12}$, where b = breadth of beam, and d its depth.

Then, substituting this value in the general formulas for beams, we have for rectangular beams of any material the following formulas: —

Beams fixed at one end, and loaded at the other (**Fig. 2**).

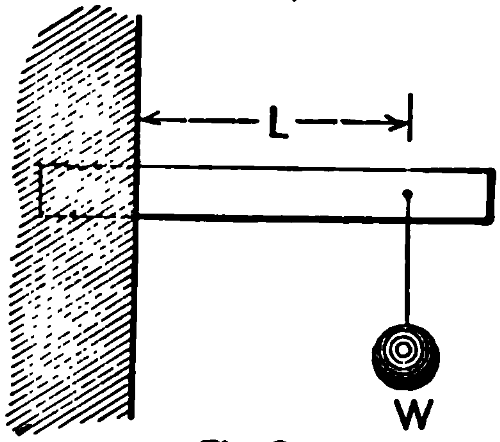


Fig. 2.

$$\text{Safe load in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A}{4 \times \text{length in feet}}, \quad (2)$$

or

$$\text{Breadth in inches} = \frac{4 \times \text{load} \times \text{length in feet}}{\text{square of depth} \times A}. \quad (3)$$

Beams fixed at one end, and loaded with uniformly distributed load (Fig. 3).

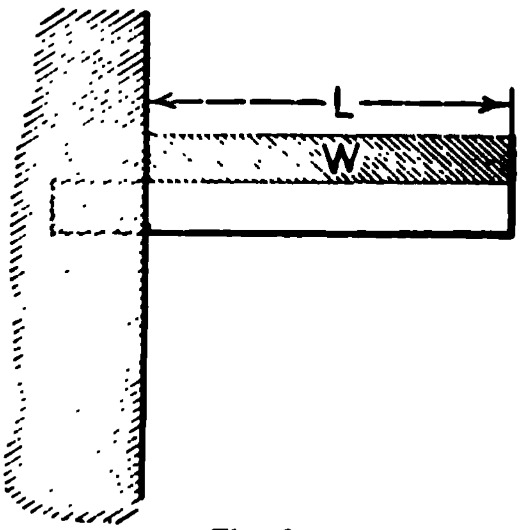


Fig 3.

$$\text{Safe load in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A}{2 \times \text{length in feet}}, \quad (4)$$

or

$$\text{Breadth in inches} = \frac{2 \times \text{length in feet} \times \text{load}}{\text{square of depth} \times A}. \quad (5)$$

Beams supported at both ends, loaded at middle **(Fig. 4).**

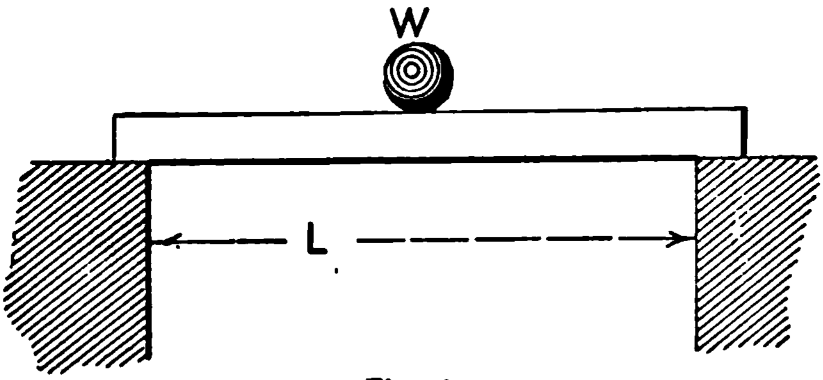


Fig 4.

$$\text{Safe load in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A}{\text{span in feet}}, \qquad (6)$$

or

$$\text{Breadth in inches} = \frac{\text{span in feet} \times \text{load}}{\text{square of depth} \times A}. \qquad (7)$$

Beams supported at both ends, load uniformly distributed **(Fig. 5).**

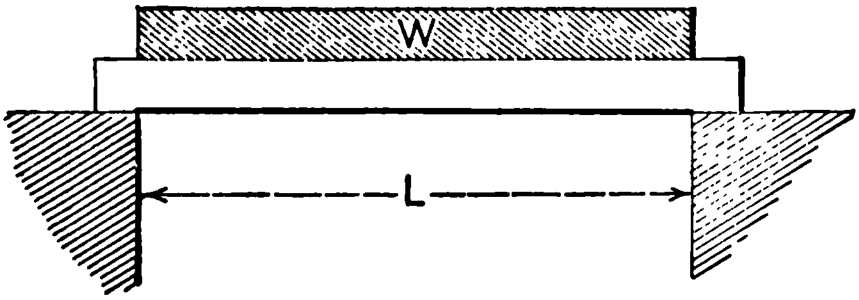


Fig. 5.

$$\text{Safe load in pounds} = \frac{2 \times \text{breadth} \times \text{square of depth} \times A}{\text{span in feet}}, \qquad (8)$$

or

$$\text{Breadth in inches} = \frac{\text{span in feet} \times \text{load}}{2 \times \text{square of depth} \times A}. \qquad (9)$$

Beams supported at both ends, loaded with concentrated load NOT AT CENTRE **(Fig. 6).**

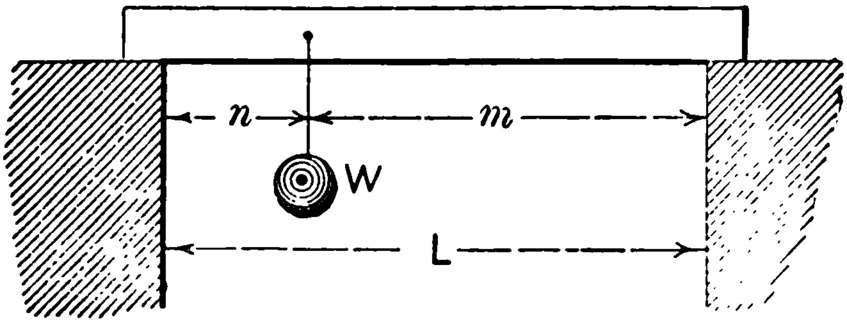


Fig 6

$$\text{Safe load in pounds} = \frac{\text{breadth} \times \text{sq. of depth} \times \text{span} \times A}{4 \times m \times n}, \qquad (10)$$

or

$$\text{Breadth in inches} = \frac{4 \times \text{load} \times m \times n}{\text{square of depth} \times \text{span} \times A}. \qquad (11)$$

Beams supported at both ends, and loaded with **W** *pounds at a distance* m *from each end* (Fig. 7).

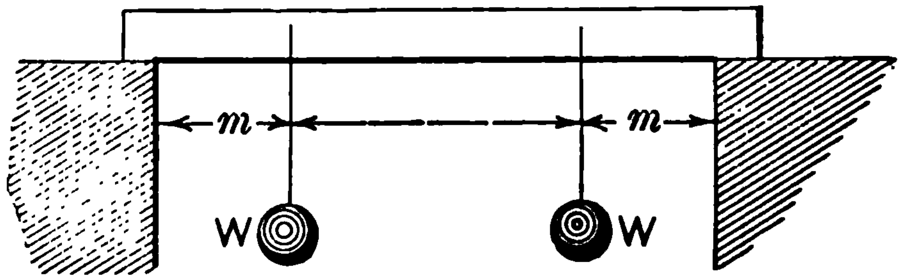


Fig. 7.

$$\text{Safe load } W \text{ in pounds at each point} = \frac{\text{breadth} \times \text{square of depth} \times A}{4 \times m}, \quad (12)$$

or

$$\text{Breadth in inches} = \frac{4 \times \text{load at one point} \times m}{\text{sq. of depth} \times A}. \quad (13)$$

NOTE.—In the last two cases the lengths denoted by m and n should be taken in feet, the same as the spans.

VALUES OF THE CONSTANT A.

The letter A denotes the safe load for a unit beam one inch square and one foot span, loaded at the centre. This is also one-eighteenth of the modulus of rupture for safe loads. The following are the values of A, which are obtained by dividing the moduli of rupture in Chap. XIV. by 18.

TABLE I.
VALUES OF A.—CO-EFFICIENT FOR BEAMS.

Material.	A lbs.	Material.	A lbs.
Cast iron	308	Pine, white, Western	65
Wrought-iron	666	" Texas yellow	90
Steel	888	Spruce	70
		Whitewood (poplar)	65
American woods:			
Chestnut	60	Bluestone flagging (Hudson River)	21
Hemlock	55	Granite, average	17
Oak, white	75	Limestone	15
Pine, Georgia yellow	100	Marble	17
" Oregon	90	Sandstone	8
" red or Norway	70	Slate	50
" white, Eastern	60		

These values for the co-efficient A are one-third of the breaking-weight of timbers of the same size and quality as that used in first-class buildings. This is a sufficient allowance for timbers in roof trusses, and beams which do not have to carry a more severe load than that on a dwelling-house floor, and small halls, etc. Where there is likely to be very much vibration, as in the floor of a mill, or a gymnasium floor, or floors of large public halls, the author recommends that only four-fifths of the above values of A be used.

Example 1.—What load will a hard-pine beam, 8 inches by 12 inches, securely fastened into a brick wall at one end, sustain with safety, 6 feet out from the wall?

Ans. Safe load in pounds (Formula 2) equals

$$\frac{8 \times 144 \times 100}{4 \times 6} = 4{,}800 \text{ lbs.}$$

Example 2.—It is desired to suspend two loads of 10,000 pounds each, 4 feet from each end of an oak beam 20 feet long. What should be the size of the beam?

Ans. Assume depth of beam to be 14 inches; then (Formula 13) breadth $= \dfrac{4 \times 10{,}000 \times 4}{196 \times 75} = 11$ inches, nearly; therefore the beam should be 11×14 inches.

Relative Strength of Rectangular Beams.

From an inspection of the foregoing formulas, it will be found that the relative strength of rectangular beams in different cases is as follows:—

Beam supported at both ends, and loaded with a uniformly distributed load	1
Beam supported at both ends, and loaded at the centre	½
Beam fixed at one end, and loaded with a uniformly distributed load	¼
Beam fixed at one end, and loaded at the other	⅛

Also the following can be shown to be true:—

Beam firmly fixed at both ends, and loaded at the centre	1
Beam fixed at both ends, and loaded with distributed load	1½

These facts are also true of a uniform beam of any form of cross-section.

When a square beam is supported on its edge, instead of on its side,—that is, has its diagonal vertical,—it will bear about seven-tenths as great a breaking-load.

The strongest beam which can be cut out of a round log is one in which the breadth is to the depth as 5 to 7, very nearly, and can be found graphically, as shown in margin. Draw any diagonal, as *ab*, and divide it into three equal parts by the points *c* and *d*; from these points draw perpendicular lines, and connect the points *e* and *f* with *a* and *b*, as shown.

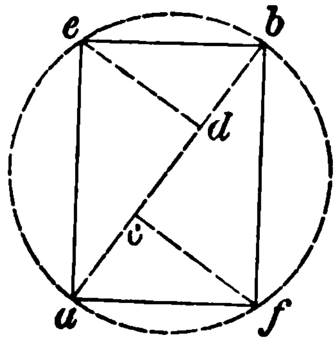


Fig. 8.

Cylindrical Beams.—A cylindrical beam is only $\frac{1}{1.7}$ as

strong as a square beam whose side is equal to the diameter of the circle. Hence, to find the load for a cylindrical beam, first find the proper load for the corresponding square beam, and then divide it by 1.7.

The bearing of the ends of a beam on a wall beyond a certain amount does not strengthen the beam any. In general, a beam should have a bearing of four inches, though, if the beam be very short, the bearing may be less.

Weight of the Beam itself to be taken into Account.—The formulas we have given for the strength of beams do not take into account the weight of the beam itself, and hence the safe load of the formulas includes both the external load and the weight of the material in the beam. In small wooden beams, the weight of the beam is generally so small, compared with the external load, that it need not be taken into account. But in larger wooden beams, and in metal and stone beams, the weight of the beam should be subtracted from the safe load if the load is distributed; and if the load is applied at the centre, one-half the weight of the beam should be subtracted.

The weight per cubic foot for different kinds of timber may be found in the table giving the *Weight of Substances*, Part III.

Tables for the strength of yellow and white pine, spruce, and oak beams, are given below, for beams one inch wide.

To find the strength of a given beam of any other breadth, it is only necessary to multiply the strength given in the table by the breadth of the given beam

EXAMPLE.—What is the safe distributed load for a yellow-pine beam, supported at both ends, 8 inches by 12 inches, 20 feet clear span?

Ans. From Table II., safe load for one inch thickness is 1,440 pounds. $1,440 \times 8 = 11,520$ pounds, safe load for beam. *For a concentrated load at centre*, divide these figures by 2.

To find the size of a beam that will support a given load with a given span, find the safe load for a beam of an assumed depth one inch wide, and divide the given load by this strength.

EXAMPLE.—What size spruce beam will be required to carry a distributed load of 8,640 pounds for a clear span of 18 feet?

Ans. From the table, we find that a beam 14 inches deep and 1 inch thick, 18 feet span, will support 1,524 pounds; and dividing the load, 8,640 pounds, by 1,524, we have $5\frac{1}{2}$ for the breadth of the ... in inches: hence the beam should be 6 by 14 inches, to carry ...ibuted load of 8,640 pounds with a span of 18 feet.

TABLE II.

HARD-PINE BEAMS.

Table of safe quiescent loads for horizontal rectangular beams of Georgia yellow pine, one inch broad, supported at both ends, load *uniformly distributed*. For *concentrated* load at centre *divide by two*. For *permanent* loads (such as masonry) reduce by 10 per cent.

Depth of beam.	Span, in feet.												
	6	8	10	12	14	15	16	17	18	20	22	24	25
ins.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
6	1,200	900	720	600	514	480	....	....	....	....	....	....	....
7	1,633	1,225	980	816	700	653	612	....	....	....	....	....	....
8	2,133	1,600	1,280	1,066	914	853	800	753	....	....	....	....	....
9	2,700	2,025	1,620	1,350	1,157	1,080	1,012	953	900	....	....	....	....
10	3,333	2,500	2,000	1,666	1,428	1,333	1,250	1,176	1,111	1,000	....	....	....
12	4,800	3,600	2,880	2,400	2,056	1,920	1,800	1,694	1,600	1,440	....	....	....
14	6,533	4,900	3,920	3,266	2,800	2,613	2,450	2,306	2,177	1,960	1,782	....	....
15	7,500	5,633	4,500	3,750	3,214	3,000	2,816	2,653	2,500	2,250	2,045	1,875	1,800
16	8,533	6,400	5,120	4,266	3,656	3,412	3,200	3,012	2,844	2,560	2,327	2,133	2,048

Loads above and to the right of heavy line will crack plastered ceilings.

For beams with a rectangular cross-section, we can simplify our formulas for strength by substituting for the moment of inertia its value, viz., $\frac{b \times d^3}{12}$, where b = breadth of beam, and d its depth.

Then, substituting this value in the general formulas for beams, we have for rectangular beams of any material the following formulas: —

Beams fixed at one end, and loaded at the other (Fig. 2).

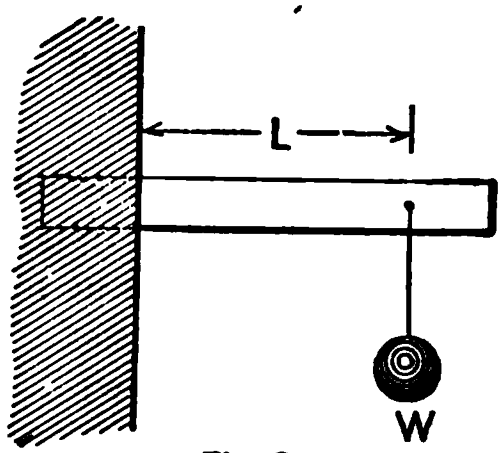


Fig. 2.

$$\text{Safe load in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A}{4 \times \text{length in feet}}, \tag{2}$$

or

$$\text{Breadth in inches} = \frac{4 \times \text{load} \times \text{length in feet}}{\text{square of depth} \times A}. \tag{3}$$

Beams fixed at one end, and loaded with uniformly distributed load (Fig. 3).

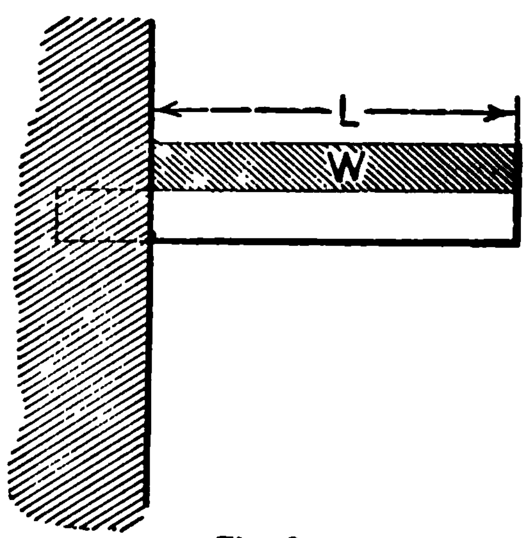


Fig 3.

$$\text{Safe load in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A}{2 \times \text{length in feet}}, \tag{4}$$

or

$$\text{Breadth in inches} = \frac{2 \times \text{length in feet} \times \text{load}}{\text{square of depth} \times A}. \tag{5}$$

***Beams supported at both ends, loaded at middle* (Fig. 4).**

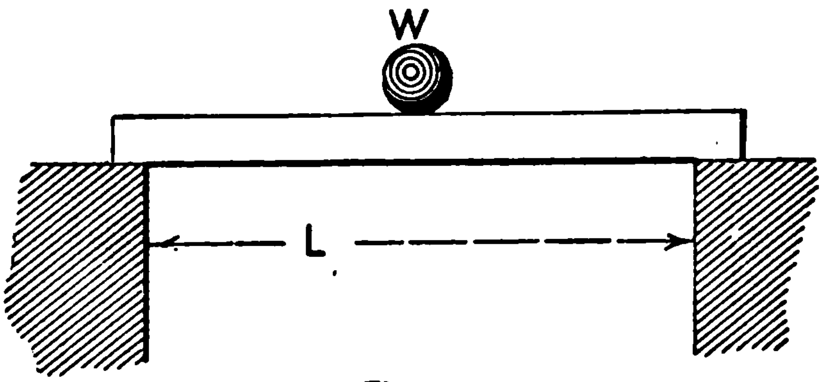


Fig 4.

$$\text{Safe load in pounds} = \frac{\text{breadth} \times \text{square of depth} \times A}{\text{span in feet}}, \quad (6)$$

r

$$\text{Breadth in inches} = \frac{\text{span in feet} \times \text{load}}{\text{square of depth} \times A}. \quad (7)$$

Beams supported at both ends, load uniformly distributed Fig. 5).

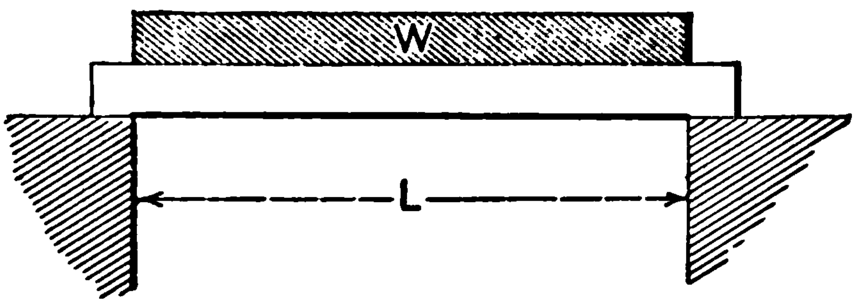


Fig. 5.

$$\text{Safe load in pounds} = \frac{2 \times \text{breadth} \times \text{square of depth} \times A}{\text{span in feet}}, \quad (8)$$

r

$$\text{Breadth in inches} = \frac{\text{span in feet} \times \text{load}}{2 \times \text{square of depth} \times A}. \quad (9)$$

Beams supported at both ends, loaded with concentrated load *VOT AT CENTRE* (Fig. 6).

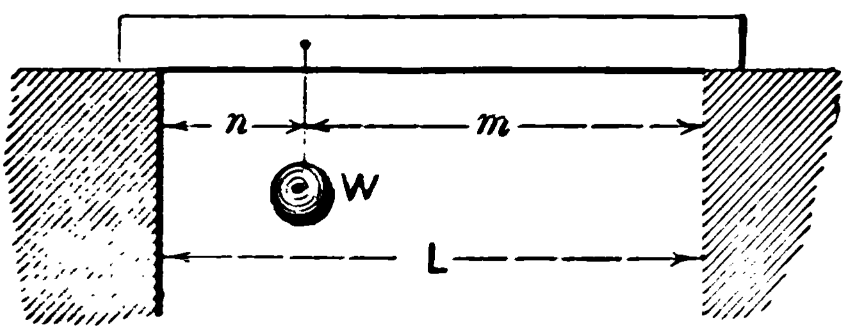


Fig 6

$$\text{Safe load in pounds} = \frac{\text{breadth} \times \text{sq. of depth} \times \text{span} \times A}{4 \times m \times n}, \quad (10)$$

or

$$\text{Breadth in inches} = \frac{4 \times \text{load} \times m \times n}{\text{square of depth} \times \text{span} \times A}. \quad (11)$$

Beams supported at both ends, and loaded with **W** *pounds at a distance* m *from each end* (Fig. 7).

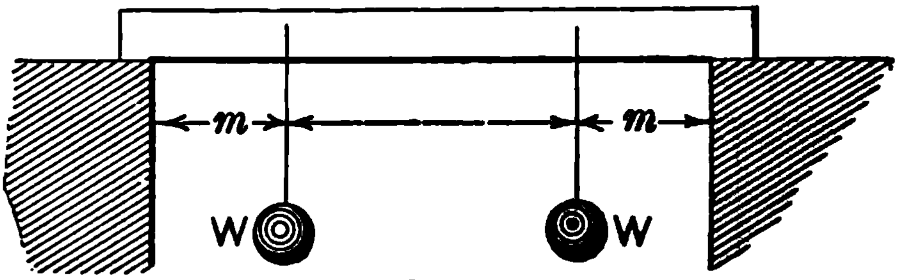


Fig. 7.

$$\text{Safe load } W \text{ in pounds at each point} = \frac{\text{breadth} \times \text{square of depth} \times A}{4 \times m}, \quad (12)$$

or

$$\text{Breadth in inches} = \frac{4 \times \text{load at one point} \times m}{\text{sq. of depth} \times A}. \quad (13)$$

NOTE.—In the last two cases the lengths denoted by m and n should be taken in feet, the same as the spans.

VALUES OF THE CONSTANT A.

The letter A denotes the safe load for a unit beam one inch square and one foot span, loaded at the centre. This is also one-eighteenth of the modulus of rupture for safe loads. The following are the values of A, which are obtained by dividing the moduli of rupture in Chap. XIV. by 18.

TABLE I.

VALUES OF A.—CO-EFFICIENT FOR BEAMS.

Material.	A lbs.	Material.	A lbs.
Cast-iron	308	Pine, white, Western	65
Wrought-iron	666	" Texas yellow	90
Steel	888	Spruce	70
		Whitewood (poplar)	65
American woods:			
Chestnut	60	Bluestone flagging (Hudson River)	21
Hemlock	55	Granite, average	17
Oak, white	75	Limestone	15
Pine, Georgia yellow	100	Marble	17
" Oregon	90	Sandstone	8
" red or Norway	70	Slate	50
" white, Eastern	60		

These values for the co-efficient A are one-third of the breaking-weight of timbers of the same size and quality as that used in first-class buildings. This is a sufficient allowance for timbers in roof trusses, and beams which do not have to carry a more severe load than that on a dwelling-house floor, and small halls, etc. Where there is likely to be very much vibration, as in the floor of a mill, a gymnasium-floor, or floors of large public halls, the author [recom]mends that only four-fifths of the above values of A be used.

Example 1.—What load will a hard-pine beam, 8 inches by 12 inches, securely fastened into a brick wall at one end, sustain with safety, 6 feet out from the wall ?

Ans. Safe load in pounds (Formula 2) equals

$$\frac{8 \times 144 \times 100}{4 \times 6} = 4{,}800 \text{ lbs.}$$

Example 2.—It is desired to suspend two loads of 10,000 pounds each, 4 feet from each end of an oak beam 20 feet long. What should be the size of the beam ?

Ans. Assume depth of beam to be 14 inches; then (Formula 13) breadth $= \frac{4 \times 10{,}000 \times 4}{196 \times 75} = 11$ inches, nearly ; therefore the beam should be 11×14 inches.

Relative Strength of Rectangular Beams.

From an inspection of the foregoing formulas, it will be found that the relative strength of rectangular beams in different cases is as follows:—

Beam supported at both ends, and loaded with a uniformly distributed load	1
Beam supported at both ends, and loaded at the centre	$\frac{1}{2}$
Beam fixed at one end, and loaded with a uniformly distributed load	$\frac{1}{4}$
Beam fixed at one end, and loaded at the other	$\frac{1}{8}$

Also the following can be shown to be true :—

Beam firmly fixed at both ends, and loaded at the centre	1
Beam fixed at both ends, and loaded with distributed load	$1\frac{1}{2}$

These facts are also true of a uniform beam of any form of cross-section.

When a square beam is supported on its edge, instead of on its side, —that is, has its diagonal vertical, —it will bear about seven-tenths as great a breaking-load.

The strongest beam which can be cut out of a round log is one in which the breadth is to the depth as 5 to 7, very nearly, and can be found graphically, as shown in margin. Draw any diagonal, as *ab*, and divide it into three equal parts by the points *c* and *d*; from these points draw perpendicular lines, and connect the points *e* and *f* with *a* and *b*, as shown.

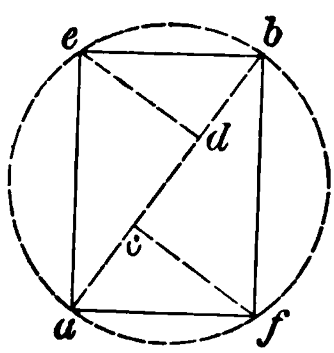


Fig. 8.

Cylindrical Beams. — A cylindrical beam is only $\frac{1}{1.7}$ as

When a beam is built of several pieces in length as well a depth, they should break joints with each other. The layers b the neutral axis should be lengthened by the scarf or fish jo used for resisting tension; and the upper ones should have the abut against each other, using plain *butt* joints.

Fig. 9.

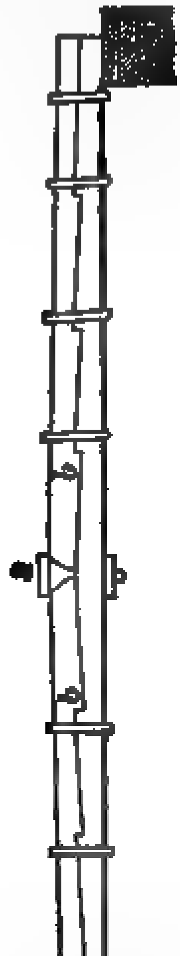


Fig. 10.

Represents a solid built beam, the top part being of two pieces, *b*, *b*, which abut against a broad flat iron belt, *a*, termed a "king bolt."

Many builders prefer using a *built beam* of selected timber single solid one, on account of the great difficulty of getting latter, when very large, free from defects: moreover, the stre of the former is to be relied upon, although it cannot be stro than the corresponding solid one, if perfectly sound.

CHAPTER XVI.

STIFFNESS AND DEFLECTION OF BEAMS.

In Chaps. XIV. and XV. we have considered the strength of beams to resist breaking only; but in all first-class buildings it is desired that those beams which show, or which support a ceiling, should not only have sufficient strength to carry the load with safety, but should do so without bending enough to present a bad appearance to the eye, or to crack the ceiling: hence, in calculating the dimensions of such beams, we should not only calculate them with regard to their resistance to breaking, but also to bending. Unfortunately, we have at present no method of combining the two calculations in one operation. A beam apportioned by the rules for strength will not bend so as to strain the fibres beyond their elastic limit, but will, in many cases, bend more than a due regard for appearance will justify.

The amount which a beam bends under a given load is called its *deflection*, and its resistance to bending is called its *stiffness:* hence the stiffness is inversely as the deflection.

The rules for the stiffness of beams are derived from those for the deflection of beams; and the latter are derived partly from mathematical reasoning, and partly from experiments.

We can find the *deflection at the centre*, of *any* beam not strained beyond the elastic limit, by the following formula: —

$$\text{Def. in inches} = \frac{\text{load in lbs.} \times \text{cube of span in inches} \times c}{\text{modulus of elasticity} \times \text{moment of inertia}}. \quad (1)$$

The values of c are as follows: —

Beam supported at both ends, loaded at centre . .	0.021
" " " uniformly loaded . .	0.013
" fixed at one end, loaded at the other	0.333
" " " uniformly loaded	0.125

By making the proper substitutions in Formula 1, we derive the

following formula for a *rectangular* beam ***supported at both ends***, and *loaded at the centre*:—

$$\text{Def. in inches} = \frac{\text{load} \times \text{cube of span} \times 1728}{4 \times \text{breadth} \times \text{cube of depth} \times E}, \quad (2)$$

the span being taken in feet. From this formula the value of the modulus of elasticity, E, for different materials, has been calculated. Thus beams of known dimensions are supported at each end, and a known weight applied at the centre of the beam. The deflection of the beam is then carefully measured; and, substituting these known quantities in Formula 2, the value of E is easily obtained.

Formula 2 may be simplified somewhat by representing $\frac{1728}{4 \times E}$ by $\frac{1}{F}$, which gives us the formula

$$\text{Def. in inches} = \frac{W \times L^3}{B \times D^3 \times F}. \quad (3)$$

For a distributed load the deflection will be five-eighths of this.

NOTE.—The constant F corresponds to Hatfield's F, in his Transverse Strains.

If we wish to find the load which shall cause a given deflection, we can transpose Formula 2 so that the load shall form the left-hand member. Thus:—

$$\begin{array}{l}\text{Load at centre}\\ \text{in pounds}\end{array} = \frac{4 \times \text{breadth} \times \text{cube of depth} \times \text{def. in ins.} \times E}{\text{cube of span} \times 1728}. \quad (4)$$

Now, that this formula may be of use in determining the load to put upon a beam, the value of the deflection must in some way be fixed. This is generally done by making it a certain proportion of the span.

Thus Tredgold and many other authorities say, that, if a floor-beam deflects more than one-fortieth of an inch for every foot of span, it is liable to crack the ceiling on the under side; and hence this is the limit which is generally given to the deflection of beams in first-class buildings.

Then, if we substitute for "deflection" the value, length in feet $\div$ 40, in the above formula, we have,

$$\text{Load at centre} = \frac{\text{breadth} \times \text{cube of depth} \times e}{\text{square of length}}, \quad (5)$$

letting $$e = \frac{E}{17280}.$$

y engineers and architects think that *one-thirtieth of an inch*)t of span is not too much to allow for the deflection of floor-

beams, as a floor is seldom subjected to its full estimated load, and then only for a short time.

If we adopt this ratio, we shall have as our constant for deflection,

$$e_1 = \frac{E}{12960}.$$

In either of the above cases, it is evident that the values used for E, F, e, or e_1, should be derived from tests on timbers of the same size and quality as those to be used. It has only been within the last three or four years that we have had any accurate tests on the strength and elasticity of large timbers, although there had been several made on small pieces of various woods.

The values of the various constants for the first three woods in the following table have been derived from tests made by Professor Lanza and his students at the Massachusetts Institute of Technology, and the values for the other woods are about six-sevenths of the values derived from Mr. Hatfield's experiments. The author believes that the values given in this table may be relied upon for timber such as is used in first-class construction.

TABLE I.

Values of Constants for Stiffness or Deflection of Beams.

E = Modulus of elasticity, pounds per square inch.

F = Constant for deflection of beam, supported at both ends, and loaded at the centre.

e = Constant, allowing a deflection of one-fortieth of an inch per foot of span.

e_1 = Constant, allowing a deflection of one-thirtieth of an inch per foot of span.

Material.	E.	$F = \frac{E}{432}$.	$e = \frac{E}{17280}$.	$e_1 = \frac{E}{12960}$.
Cast iron	15,700,000	36,300	907	1210
Wrought-iron	26,000,000	60,000	1500	2000
Steel	31,000,000	71,760	1794	2358
Yellow pine	1,780,000	4,120	103	137
Spruce	1,294,000	3,000	75	100
White oak	1,240,000	2,870	72	95
White pine	1,073,000	2,480	62	82
Hemlock	1,045,000	2,420	60	80
Whitewood	1,278,000	2,960	74	98
Chestnut	944,000	2,180	54	72
Ash	1,482,000	3,430	86	114
Maple	1,902,000	4,400	110	146

Continuous Girder of Three Equal Spans, Concentrated Load of W *Pounds at Centre of Each Span.*

Re-action of either abutment,

$$R_1 = R_4 = \tfrac{7}{20} W; \qquad (7)$$

Re-action of either central support,

$$R_2 = R_3 = \tfrac{23}{20} W; \qquad (8)$$

or the re-action of the end supports is lessened three-tenths, and that of the central supports increased three-twentieths, of that which they would have been, had three separate girders of the same cross-section been used, instead of one continuous girder.

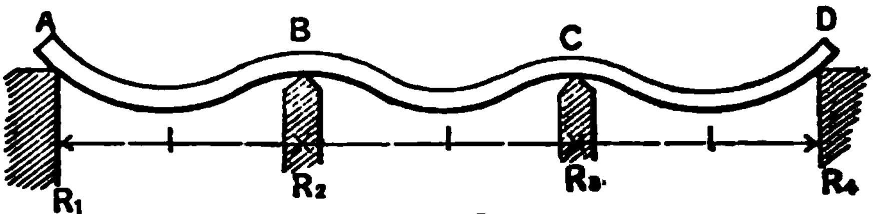


Fig.2

Continuous Girder of Three Equal Spans uniformly loaded with w *Pounds per Unit of Length.*

Re-action of either end support,

$$R_1 = R_4 = \tfrac{2}{5} wl; \qquad (9)$$

Re-action of either central support,

$$R_2 = R_3 = \tfrac{11}{10} wl; \qquad (10)$$

hence the re-actions of the end supports are one-fifth less, and of the central supports one-tenth more, than if the girder were not continuous.

Strength of Continuous Girders. — Having determined the re-action of the supports, we will now consider the strength of the girder.

The strength of a beam depends upon the material and shape of the beam, and upon the external conditions imposed upon the beam. The latter give rise to the bending-moment of the beam, or the amount by which the external forces (such as the load and supporting forces) tend to bend and break the beam.

It is this bending-moment which causes the difference in the bearing-strength of continuous and non-continuous girders of the same cross-section.

Continuous Girders of Two Spans. — When a rectangular beam is at the point of breaking, we have the following conditions : —

$$\text{Bending-moment} = \frac{\text{Mod. of rupture} \times \text{breadth} \times \text{sq. of depth}}{6}; \qquad (11)$$

:hat the beam may carry its load with perfect safety, we must the load by a proper factor of safety.

Hence, if we can determine the bending-moment of a beam under any conditions, we can easily determine the required dimensions of the beam from Formula 11.

The greatest bending-moment for a continuous girder of two spans is almost always over the middle support, and is of the opposite kind to that which tends to break an ordinary beam.

Distributed Load. — The greatest bending-moment in a continuous girder of two spans, l and l_1, loaded with a uniformly distributed load of w pounds per unit of length, is

$$\text{Bending-moment} = \frac{wl^3 + wl_1^3}{8(l + l_1)}. \tag{12}$$

When $l = l_1$, or both spans are equal,

$$\text{Bending-moment} = \frac{wl^2}{8}, \tag{12a}$$

which is the same as the bending-moment of a beam supported at both ends, and uniformly loaded over its whole length: hence *a continuous girder of two spans uniformly loaded is no stronger than if non-continuous.*

Concentrated Load. — The greatest bending-moment in a continuous girder of two equal spans, each of length l, loaded with W pounds at centre of one span, and with W_1 pounds at the centre of the other span, is

$$\text{Bending-moment} = \tfrac{3}{32}\, l\,(W + W_1). \tag{13}$$

When $W = W_1$, or the two loads are equal, this becomes

$$\text{Bending-moment} = \tfrac{3}{16}\, Wl, \tag{13a}$$

or one-fourth less than what it would be were the beam cut at the middle support.

Continuous Girder of Three Spans, Distributed Load. — The greatest bending-moment in a continuous girder of three spans loaded with a uniformly distributed load of w pounds per unit of length, the length of each end span being l_1 and of the middle span l, is at either of the central supports, and is represented by the formula,

$$\text{Bending-moment} = \frac{wl^3 + wl_1^3}{4(3l + 2l_1)}. \tag{14}$$

When the three spans are equal, this becomes

$$\text{Bending-moment} = \frac{wl^2}{10}, \tag{14a}$$

or one-fifth less than what it would be were the beam not continuous.

TABLE II.

HARD-PINE BEAMS.

Table of maximum distributed loads which can be supported by horizontal rectangular beams of Georgia yellow pine, one inch broad, and supported at both ends, with safety and without deflecting more than one-thirtieth of an inch per foot of span.

Depth of beam.	Span. 4 feet.	Span. 6 feet.	Span. 8 feet.	Span. 10 feet.	Span. 12 feet.	Span. 14 feet.	Span. 16 feet.	Span. 18 feet.	Span. 20 feet.	Span. 22 feet.	Span. 24 feet.	Depth of beam.
ins.	lbs.	lbs.	lbs.	lbs.	lbs	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	ins.
6	1,800	1,200	738	473	328	242	185	146	118	97	82	6
8	3,200	2,133	1,600	1,121	778	573	438	346	280	231	194	8
9	4,050	2,700	2,025	1,506	1,108	816	624	493	399	329	277	9
10	5,000	3,333	2,500	2,000	1,520	1,120	856	676	548	452	380	10
12	7,200	4,800	3,600	2,880	2,400	1,985	1,479	1,168	950	781	656	12
14	9,800	6,533	4,900	3,920	3,266	2,800	2,348	1,855	1,503	1,240	1,042	14
15	11,250	7,500	5,633	4,500	3,750	3,214	2,816	2,281	1,850	1,525	1,282	15
16	12,800	8,533	6,400	5,120	4,266	3,656	3,200	2,769	2,244	1,851	1,536	16

Loads *above* horizontal lines are calculated by formula for *stiffness*; those *below*, by formula for *strength*.

SPRUCE BEAMS.

Table of maximum distributed loads which can be supported by horizontal rectangular beams of spruce timber, one inch broad, and supported at both ends, with safety and without deflecting more than one-thirtieth of an inch per foot of span.

Depth of beam.	Span, 4 feet.	Span, 6 feet.	Span, 8 feet.	Span, 10 feet.	Span, 12 feet.	Span, 14 feet.	Span, 16 feet.	Span, 18 feet.	Span, 20 feet.	Span, 22 feet.	Span, 24 feet.	Depth of beam.
ins.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	ins.
6	1,260	840	540	345	240	176	135	106	86	71	60	6
8	2,240	1,493	1,120	819	569	417	320	251	205	170	142	8
9	2,834	1,890	1,417	1,134	810	584	455	359	292	240	202	9
10	3,500	2,333	1,750	1,400	1,111	816	625	493	400	390	277	10
12	5,040	3,360	2,520	2,016	1,680	1,410	1,060	851	691	570	480	12
14	6,860	4,573	3,430	2,744	2,286	1,960	1,715	1,352	1,098	905	762	14
15	7,874	5,250	3,937	3,150	2,625	1,875	1,968	1,663	1,350	1,114	935	15
16	8,960	5,973	4,480	3,584	2,986	2,540	2,240	2,108	1,688	1,352	1,134	16

Loads *above* horizontal lines are calculated by formula for *stiffness*; those *below*, by formula for *strength*.

TABLE IV.

OAK BEAMS.

Table of maximum distributed loads which can be supported by horizontal rectangular beams of white oak, one inch broad, and supported at both ends, with safety and without deflecting more than one-thirtieth of an inch per foot of span.

Depth of beam.	Span, 4 feet.	Span, 6 feet.	Span, 8 feet.	Span, 10 feet.	Span, 12 feet.	Span, 14 feet.	Span, 16 feet.	Span, 18 feet.	Span, 20 feet.	Span, 22 feet.	Span, 24 feet.	Depth of beam.
ins.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	ins.
6	1,350	900	511	328	226	167	128	101	82	68	57	6
8	2,400	1,600	1,213	778	537	397	303	240	194	160	135	8
9	3,038	2,025	1,519	1,108	765	565	432	342	277	229	192	9
10	3,750	2,500	1,875	1,500	1,050	775	593	470	380	314	264	10
12	5,400	3,600	2,700	2,160	1,800	1,339	1,024	812	656	542	456	12
14	7,350	4,900	3,675	2,940	2,450	2,100	1,627	1,289	1,042	862	724	14
15	8,436	5,625	4,218	3,375	2,812	2,410	2,001	1,586	1,282	1,060	891	15
16	9,600	6,400	4,800	3,840	3,200	2,742	2,560	1,925	1,556	1,286	1,081	16

Loads *above* horizontal lines are calculated by formula for *stiffness*; those *below*, by formula for *strength*.

EXAMPLE 2. — What should be the dimensions of a yellow-pine beam of 10 foot span, to support a concentrated load of 4250 pounds, without deflecting more than $\frac{1}{2}$ of an inch at the centre?

Ans. A deflection of $\frac{1}{2}$ of an inch in a span of 10 feet is in the proportion of $\frac{1}{20}$ of an inch per foot of span; and as the load is concentrated, and applied at the centre, we should use Formula 7, employing for e the value given in the fourth column, opposite yellow pine.

Formula 7 gives the dimensions of the breadth, and to obtain it we must assume a value for the depth. For this we will first try 8 inches.

Substituting in Formula 7, we have,

$$\text{Breadth} = \frac{4250 \times 100}{512 \times 137} = 6 \text{ inches, nearly.}$$

This would give us a beam 6 by 8 inches.

EXAMPLE 3. — What is the largest load that an inclined spruce beam 8 by 12 inches, 12 feet long between supports, will carry at the centre, consistent with stiffness, the horizontal distance between the supports being 10 feet?

Ans. Formula 12 is the one to be employed; and we will use the value of e given in the third column, opposite spruce. Making the proper substitutions, we have,

$$\text{Safe load} = \frac{8 \times 1728 \times 75}{12 \times 10} = 8640 \text{ pounds.}$$

Cylindrical Beams.

For cylindrical beams the same formulas may be employed as for rectangular beams, only, instead of e, use $1.7 \times e$; that is, a cylindrical beam bends 1.7 times as much as the circumscribing rectangle.

Deflection of Iron Beams.

For rolled-iron beams the deflection is most accurately obtained by Formula 1. The following approximate formula gives the deflections quite accurately for the maximum safe loads,

$$\text{Deflection in inches} = \frac{\text{square of span in feet}}{70 \times \text{the depth of beam}}.$$

The deflections for the *Phœnix*, *Pencoyd*, *Trenton*, and *Carnegie* beams, are given in the tables for strength of beams, in Chap. XIV.

In using iron beams, it should be remembered that the *deepest beam* is always the most economical; and the stiffness of a floor is always greater when a suitable number of deep beams are used.

CHAPTER XVII.

STRENGTH AND STIFFNESS OF CONTINUOUS GIRDERS

GIRDERS resting upon three or more supports are of quite frequent occurrence in building construction; and a great variety of opinions is held as to the relative strength and stiffness of continuous and non-continuous girders; very few persons, probably, having any correct knowledge of the subject.

In almost every building of importance, it is necessary to employ girders resting upon piers or columns placed from eight to fifteen feet apart; and in many cases girders can conveniently be obtained which will span two and even three of the spaces between the piers or columns. When this is the case, the question arises, whether it will be better construction to use a long continuous girder, or to have each girder of only one span.

Most architects are probably aware that a girder of two or more spans is stronger and stiffer than a girder of the same section, of only one span, but just *how much* stronger and stiffer is a question they are unable to answer.

As it is seldom that a girder of more than three spans is employed in ordinary buildings, we shall consider only these two cases. In all structures, the first point which should be considered is the resistance required of the supports, and we will first consider the resistance offered by the supports of a continuous girder.

In this chapter we shall not go into the mathematical discussion of the subject, but refer any readers interested in the derivation of the formulas for continuous girders to an article on that subject, by the author, in the July (1881) number of Van Nostrand's "Engineering Magazine."

Supporting Forces.

Girders of Two Spans, loaded at the Centre of Each Span. — If a girder of two spans, l and l_1, is loaded at the centre of the span l

with W pounds, and at the centre of l_1 with W_1 pounds, the re-action of the support R_1 will be represented by the formula

$$R_1 = \frac{13W - 3W_1}{32}, \tag{1}$$

the re-action of the support R_2 by

$$R_2 = \frac{11}{16}(W + W_1), \tag{2}$$

and the re-action of the support R_3 by the formula

$$R_3 = \frac{13W_1 - 3W}{32}. \tag{3}$$

If $W = W_1$, then each of the end supports would have to sustain $\frac{5}{16}$ of one of the loads, and the centre support $\frac{11}{8}$ of W. Were the girder cut so as to make two girders of one span each, then the end supports would carry $\frac{1}{2}$ or $\frac{8}{16}W$, and the centre support $\frac{16}{16}W$: hence we see, that, by having the girder continuous, we do not require so much resistance from the end supports, but more from the central support.

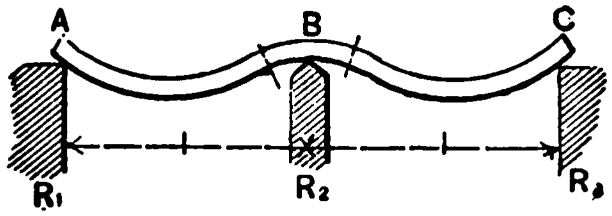


Fig 1.

Girder of Two Spans, uniformly Distributed Load over Each Span. — Load over each span equals w pounds per unit of length. Re-action of left support,

$$R_1 = \frac{w}{2}\left[l - \frac{l_1^3 + l^3}{4l(l + l_1)}\right]. \tag{4}$$

Re-action of central support,

$$R_2 = w(l + l_1) - R_1 - R_3. \tag{5}$$

Re-action of right support,

$$R_3 = \frac{w}{2}\left[l_1 - \frac{l_1^3 + l^3}{4l_1(l + l_1)}\right]. \tag{6}$$

When both spans are equal to l, the re-action of each end support is $\frac{3}{8}wl_1$, and of the central support $\frac{5}{4}wl$: hence the girder, by being continuous, reduces the re-action of the end supports, and increases that of the central support by one-fourth, or twenty-five per cent.

Continuous Girder of Three Equal Spans, Concentrated Load of **W** *Pounds at Centre of Each Span.*

Re-action of either abutment,

$$R_1 = R_4 = \tfrac{7}{20}W; \tag{7}$$

Re-action of either central support,

$$R_2 = R_3 = \tfrac{23}{20}W; \tag{8}$$

or the re-action of the end supports is lessened three-tenths, and that of the central supports increased three-twentieths, of that which they would have been, had three separate girders of the same cross-section been used, instead of one continuous girder.

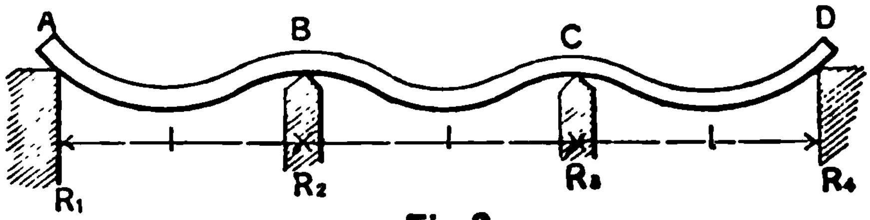


Fig.2

Continuous Girder of Three Equal Spans uniformly loaded with w *Pounds per Unit of Length.*

Re-action of either end support,

$$R_1 = R_4 = \tfrac{2}{5}wl; \tag{9}$$

Re-action of either central support,

$$R_2 = R_3 = \tfrac{11}{10}wl; \tag{10}$$

hence the re-actions of the end supports are one-fifth less, and of the central supports one-tenth more, than if the girder were not continuous.

Strength of Continuous Girders. — Having determined the re-action of the supports, we will now consider the strength of the girder.

The strength of a beam depends upon the material and shape of the beam, and upon the external conditions imposed upon the beam. The latter give rise to the bending-moment of the beam, or the amount by which the external forces (such as the load and supporting forces) tend to bend and break the beam.

It is this bending-moment which causes the difference in the bearing-strength of continuous and non-continuous girders of the same cross-section.

Continuous Girders of Two Spans. — When a rectangular beam is at the point of breaking, we have the following conditions: —

$$\text{Bending-moment} = \frac{\text{Mod. of rupture} \times \text{breadth} \times \text{sq. of depth}}{6}; \tag{11}$$

and, that the beam may carry its load with perfect safety, we must divide the load by a proper factor of safety.

Hence, if we can determine the bending-moment of a beam under any conditions, we can easily determine the required dimensions of the beam from Formula 11.

The greatest bending-moment for a continuous girder of two spans is almost always over the middle support, and is of the opposite kind to that which tends to break an ordinary beam.

Distributed Load. — The greatest bending-moment in a continuous girder of two spans, l and l_1, loaded with a uniformly distributed load of w pounds per unit of length, is

$$\text{Bending-moment} = \frac{wl^3 + wl_1^3}{8(l + l_1)}. \tag{12}$$

When $l = l_1$, or both spans are equal,

$$\text{Bending-moment} = \frac{wl^2}{8}, \tag{12a}$$

which is the same as the bending-moment of a beam supported at both ends, and uniformly loaded over its whole length: hence *a continuous girder of two spans uniformly loaded is no stronger than if non-continuous.*

Concentrated Load. — The greatest bending-moment in a continuous girder of two equal spans, each of length l, loaded with W pounds at centre of one span, and with W_1 pounds at the centre of the other span, is

$$\text{Bending-moment} = \tfrac{3}{32}\, l(W + W_1). \tag{13}$$

When $W = W_1$, or the two loads are equal, this becomes

$$\text{Bending-moment} = \tfrac{3}{16}\, Wl, \tag{13a}$$

or one-fourth less than what it would be were the beam cut at the middle support.

Continuous Girder of Three Spans, Distributed Load. — The greatest bending-moment in a continuous girder of three spans loaded with a uniformly distributed load of w pounds per unit of length, the length of each end span being l_1 and of the middle span l, is at either of the central supports, and is represented by the formula,

$$\text{Bending-moment} = \frac{wl^3 + wl_1^3}{4(3l + 2l_1)}. \tag{14}$$

When the three spans are equal, this becomes

$$\text{Bending-moment} = \frac{wl^2}{10}, \tag{14a}$$

or one-fifth less than what it would be were the beam not continuous.

Concentrated Loads. — The greatest bending-moment in a continuous girder of three equal spans, each of a length l, and each loaded at the centre with W pounds, is

$$\text{Bending-moment} = \tfrac{3}{20} W l, \tag{15}$$

or two-fifths less than that of a non-continuous girder.

Deflection of Continuous Girders.

Continuous Girder of Two Equal Spans. — The greatest deflection of a continuous girder of two equal spans, loaded with a uniformly distributed load of w pounds per unit of length, is

$$\text{Deflection} = 0.005416 \frac{wl^4}{EI}. \tag{16}$$

(E denotes modulus of elasticity; I, moment of inertia.)

The deflection of a similar beam supported at both ends, and uniformly loaded, is

$$\text{Deflection} = 0.013020 \frac{wl^4}{EI}.$$

Hence the deflection of the continuous girder is only about two-fifths that of a non-continuous girder. The greatest deflection in a continuous girder is also not at the centre of either span, but between the centre and the abutments.

The greatest deflection of a continuous girder of two equal spans, loaded at the centre of one span with a load of W pounds, and at the centre of the other span with W_1 pounds, is, for the span with load W,

$$\text{Deflection} = \frac{(23 W - 9 W_1) l^3}{1536 EI}; \tag{17}$$

for the span with load W_1,

$$\text{Deflection} = \frac{(23 W_1 - 9 W) l^3}{1536 EI}. \tag{17a}$$

When both spans have the same load,

$$\text{Deflection} = \frac{7}{768} \frac{W l^3}{EI}. \tag{17b}$$

The deflection of a beam supported at both ends, and loaded at the centre with W pounds, is

$$\text{Deflection} = \frac{W l^3}{48 EI};$$

or the deflection of the continuous girder is only seven-sixteenths of the non-continuous one.

Continuous Girder of Three Equal Spans. — Uniformly distributed load of w pounds per unit of length,

$$\text{Deflection at centre of middle span} = 0.00052\,\frac{wl^4}{EI} \qquad (18)$$

$$\text{Greatest deflection in end spans} = 0.006884\,\frac{wl^4}{EI} \qquad (19)$$

or the greatest deflection in the girder is only about one-half that of a non-continuous girder.

Concentrated load of W pounds at centre of each span,

$$\text{Deflection at centre of middle span} = \frac{1}{480}\,\frac{Wl^3}{EI} \qquad (20)$$

$$\text{Deflection at centre of end spans} = \frac{11}{960}\,\frac{Wl^3}{EI} \qquad (21)$$

or only eleven-twentieths of the non-continuous girder.

Several Observations and Formulas for Designing Continuous Girders.

From the foregoing we can draw many observations and conclusions, which will be of great use in deciding whether it will be best in any given case to use a continuous or non-continuous girder.

First as to the Supports. — We see from the formulas given for the re-action of the supporting forces in the different cases, that in all cases the end supports do not have as much load brought upon them when the girder is continuous as when it is not; but of course the difference must be made up by the other supports. This might often be desirable in buildings where the girders run across the building, the ends resting on the side walls, and the girders being supported at intermediate points by columns or piers. In such a case, by using a continuous girder, part of the load could be taken from the walls, and transferred to the columns or piers.

But there is another question to be considered in such a case, and that is vibration. Should the building be a mill or factory in which the girders had to support machines, then any vibration given to the middle span of the beam would be carried to the side walls if the beam were continuous, while if separate girders were used, with their ends an inch or so apart, but little if any vibration would be carried to the side walls from the middle span.

In all cases of important construction, the supporting forces should be carefully looked after.

Strength. — As the relative strength of continuous and non-continuous girders of the same size and span, and loaded in the same way, is as their bending-moments, we can easily calculate the

strength of a continuous girder, knowing the formulas for its bending-moment. From the values given for the bending-moments of the various cases considered, we see that the portion of the girder most strained is that which comes over the middle supports; also that, except in the single case of a girder of two spans uniformly loaded, the strength of a girder is greater if it is continuous than if it is not. But the gain in strength in some instances is not very great, although it is generally enough to pay for making the girder continuous.

Stiffness. — The stiffness of a girder is indirectly proportional to its deflection; that is, the less the deflection under a given load, the stiffer the girder.

Now, from the values given for the deflection of continuous girders, we see that a girder is rendered very much stiffer by being made continuous; and this may be considered as the principal advantage in the use of such girders.

It is often the case in building-construction, that it is necessary to use beams of much greater strength than is required to carry the superimposed load, because the deflections would be too great if the beam were made smaller. But, if we can use continuous girders, we may make the beams of just the size required for strength; as the deflections will be lessened by the fact of the girders being continuous. It should therefore be remembered, that, where great stiffness is required, continuous beams or girders should be used if possible.

Formulas for Strength and Stiffness.

For convenience we will give the proper formulas for calculating the strength and stiffness of continuous girders of rectangular cross-section. The formulas for strength are deduced from the formula,

$$\text{Bending-moment} = \frac{B \times D^2 \times R}{6}, \tag{22}$$

where R is a constant known as the modulus of rupture, and is eighteen times what is generally known as the co-efficient of strength.

STRENGTH. — *Continuous girder of* TWO *equal spans, loaded uniformly over each span,*

$$\text{Breaking-weight} = \frac{2 \times B \times D^2 \times A}{L}, \tag{23}$$

where B denotes the breadth of the girder, D the depth of the girder (both in inches), and L the length of one span, *in feet*. The

values of the constant A are three times the values given in Table I., p. 374. For yellow pine, 300 pounds; for spruce, 210 pounds; and for white pine, 180 pounds, — may be taken as reliable values for A.

Continuous girder of TWO *equal spans, loaded equally at the centre of each span,*

$$\text{Breaking-weight} = \frac{4}{3} \times \frac{B \times D^2 \times A}{L}. \tag{24}$$

Continuous girder of THREE *equal spans, loaded uniformly over each span,*

$$\text{Breaking-weight} = \frac{5}{2} \times \frac{B \times D^2 \times A}{L}. \tag{25}$$

Continuous girder of THREE *equal spans, loaded equally at the centre of each span,*

$$\text{Breaking-weight} = \frac{5}{3} \times \frac{B \times D^2 \times A}{L}. \tag{26}$$

STIFFNESS. — The following formulas give the loads which the beams will support without deflecting more than *one-thirtieth* of an inch per foot of span.

Continuous girder of TWO *equal spans, loaded uniformly over each span,*

$$\text{Load on one span} = \frac{B \times D^3 \times e}{0.26 \times L^2}. \tag{27}$$

Continuous girder of TWO *equal spans, loaded equally at centre of each span,*

$$\text{Load on one span} = \frac{16}{7} \times \frac{B \times D^3 \times e}{L^2}. \tag{28}$$

Continuous girder of THREE *equal spans, loaded uniformly over each span,*

$$\text{Load on one span} = \frac{B \times D^3 \times e}{0.33 \times L^2}. \tag{29}$$

Continuous girder of THREE *equal spans, loaded equally at the centre of each span,*

$$\text{Load on one span} = \frac{20}{11} \times \frac{B \times D^3 \times e}{L^2}. \tag{30}$$

The value of the constant e is obtained by dividing the modulus of elasticity by 12,960; and, for the three woods most commonly used as beams, the following values may be taken: —

Yellow pine, 187; white pine, 82; spruce, 100.

For iron beams we may find the bending-moment by the formulas given, and, from tables giving the strength and sections of rolled beams, find the beam whose moment of inertia =

$$\frac{\text{bending-moment} \times \text{depth of beam}}{2000}$$

when the bending moment is in foot pounds.

For example, we have a continuous I-beam of three equal spans, loaded over each span, with 2000 pounds per foot, distributed. Each span being 10 feet, then, from formula 14*a*, we have

$$\text{Bending-moment} = \frac{2000 \times 100}{10} = 20000.$$

$$\text{Moment of inertia} = \frac{20000}{2000} \times \text{depth of beam};$$

$20,000 \div 2000 = 10$, and we must find a beam whose depth multiplied by ten will equal its moment of inertia.

If we try a ten-inch beam, we should have $10 \times 10 = 100$; and we see from Tables, pp. 260–272, that no ten-inch beam has a moment of inertia as small as 100: so we will take a nine-inch beam. $9 \times 10 = 90$, and the lightest nine-inch beam has a moment of inertia of 93: so we will use that beam. In the case of continuous I-beams of three equal spans, equally loaded with a distributed load, we may take four-fifths of the load on one span, and find the iron beam which would support that load if with only one span.

EXAMPLE. — If we have an I-beam of three equal spans of 10 feet each loaded with 20,000 pounds over each span, what size beam should we use?

Ans. $\frac{4}{5}$ of $20,000 = 16,000$. The equivalent load for a span of one foot would be $16,000 \times 10 = 160,000$.

From Tables, Chap. XIV., we find that the beam whose co-efficient is nearest to this is the nine-inch light beam, — the same beam which we found to carry the same load in the preceding example. For beams of two equal spans loaded uniformly, the strength of the beam is the same as though the beam were not continuous.

The formulas given for the re-actions of the supports and for the deflections of continuous girders with concentrated loads, were verified by the author by means of careful experiments on small steel bars. The other formulas have been verified by comparison with other authorities, where it was possible to do so; though one or two of the cases given, the author has never seen discussed in any work on the subject.

CHAPTER XVIII.

FLITCH PLATE GIRDERS.

In framing large buildings, it often occurs that the floors must be supported upon girders, which themselves rest upon columns; and it is required that the columns shall be spaced farther apart than would be allowable if wooden girders were used. In such cases the Flitch Plate girder may be used, oftentimes with advantage. A section and elevation of a Flitch Plate girder is shown in Fig. 1.

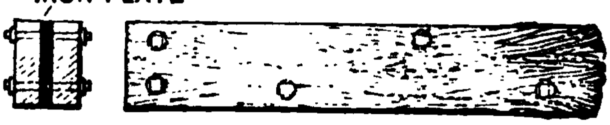


Fig. 1.

The different pieces are bolted together every two feet by three-fourths-inch bolts, as shown in elevation. It has been found by practice that the thickness of the iron plate should be about one-twelfth of the whole thickness of the beam, or the thickness of the wood should be eleven times the thickness of the iron. As the elasticity of iron is so much greater than that of wood, we must proportion the load on the wood so that it shall bend the same amount as the iron plate: otherwise the whole strain might be thrown on the iron plate. The modulus of elasticity of wrought-iron is about thirteen times that of hard pine; or a beam of hard pine one inch wide would bend thirteen times as much as a plate of iron of the same size under the same load. Hence, if we want the hard-pine beam to bend the same as the iron plate, we must put only one-thirteenth as much load on it. If the wooden beam is eleven times as thick as the iron one, we should put eleven-thirteenths of its safe load on it, or, what amounts to the same thing, use a constant only eleven-thirteenths of the strength of the wood. On this basis the following formulas have been made up for the strength of Flitch Plate girders, in which the thickness of the iron is one-twelfth of the breadth of the beam, approximately :—

Let D = Depth of beam.
B = Total thickness of wood.
L = Clear span in feet.
t = Thickness of iron plate.
$f = \begin{cases} 100 \text{ pounds for hard pine.} \\ 73 \text{ pounds for spruce.} \end{cases}$
W = Total load on girder.

Then, for beams supported at both ends,

Safe load at centre, in pounds $= \dfrac{D^2}{L}(fB + 750t)$. (1)

Safe distributed load, in pounds $= \dfrac{2D^2}{L}(fB + 750t)$. (2)

For distributed load, $D = \sqrt{\dfrac{WL}{2fB + 1500t}}$. (3)

For load at centre $D = \sqrt{\dfrac{WL}{fB + 750t}}$. (4)

As an example of the use of this kind of girder, we will take the case of a railway-station in which the second story is devoted to offices, and where we must use girders to support the second floor, of twenty-five feet span, and not less than twelve feet on centres, if we can avoid it. This would give us a floor area to be supported by the girder of 12 × 25 = 300 square feet; and, allowing 105 pounds per square foot as the weight of the superimposed load and of the floor itself, we have 31,500 pounds as the load to be supported by the girder. Now we find, by computation, that if we were to use a solid girder of hard pine, it would require a seventeen-inch by fourteen-inch beam. If we were to use an iron beam, we find that a fifteen-inch heavy iron beam would not have the requisite strength for this span, and that we should be obliged to use two twelve-inch beams.

We will now see what size of Flitch Plate girder we would require, should we decide to use such a girder. We will assume the total breadth of both beams to be twelve inches, so that we can use two six-inch timbers, which we will have hard pine. The thickness of the iron will be one inch and one-eighth. Then, substituting in Formula 3, we have,

$$D = \sqrt{\frac{31500 \times 25}{2 \times 100 \times 12 + 1500 \times 1\frac{1}{8}}} = \sqrt{192}, \text{ or 14 inches.}$$

Hence we shall require a twelve-inch by fourteen-inch girder. Now,

for a comparison of the cost of the three girders we have considered in this example. The seventeen-inch by fourteen-inch hard-pine girder would contain 515 feet, board measure, which, at five cents a foot, would amount to $25.75.

Two twelve-inch iron beams 25 feet 8 inches long will weigh 2083 pounds; and, at four cents a pound, they would cost $83.32. The Flitch-Plate girder would contain 364 feet, board measure, which would cost $18.20. The iron plate would weigh 1312½ pounds, which would cost $52.50; making the total cost of the girder $70.70, or $13 less than the iron beams, and $45 more than the solid hard-pine beams. Flitch-Plate beams also possess the advantage that the wood almost entirely protects the iron; so that, in case of a fire, the heat would not probably affect the iron until the wooden beams were badly burned.

CHAPTER XIX.

TRUSSED BEAMS.

WHENEVER we wish to support a floor upon girders having a span of more than thirty feet, we must use either a trussed girder, a riveted iron-plate girder, or two or more iron beams. The cheapest and most convenient way is, probably, to use a large wooden girder, and truss it, either as in Figs. 1 and 2, or Figs. 3 and 4.

In all these forms, it is desirable to give the girders as much depth as the conditions of the case will permit; as, the deeper the girder, the less strain there is in the pieces.

In the belly-rod truss we either have two beams, and one rod which runs up between them at the ends, or three beams, and two rods running up between the beams in the same way. The beams should be in one continuous length for the whole span of the girder, if they can be obtained that length. The requisite dimensions of the tie-rod, struts, and beam, in any given case, must be determined by first finding the stresses which come upon these pieces, and then the area of cross-section required to resist these stresses. FOR SINGLE STRUT BELLY-ROD TRUSSES, such as is represented by Fig. 1, the strain upon the pieces may be obtained by the following formulas: —

For DISTRIBUTED LOAD W *over whole girder,*

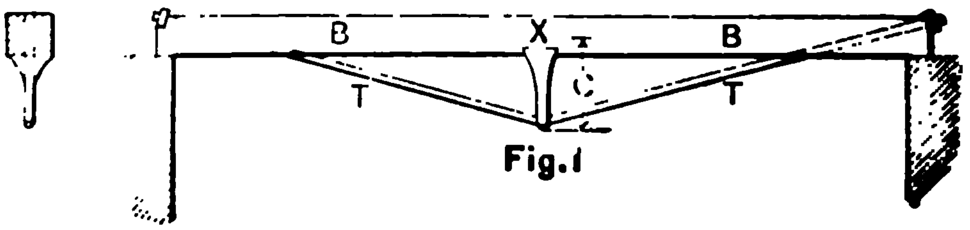


Fig. 1

$$\text{Tension in } T \quad = \frac{3}{10} W \times \frac{\text{length of } T}{\text{length of } C} \tag{1}$$

$$\text{Compression in } C = \frac{5}{8} W. \tag{2}$$

$$\text{Compression in } B = \frac{3}{10} W \times \frac{\text{length of } B}{\text{length of } C} \tag{3}$$

For CONCENTRATED LOAD *W over C*,

$$\text{Tension in } T \quad = \frac{W}{2} \times \frac{\text{length of } T}{\text{length of } C}. \tag{4}$$

$$\text{Compression in } C = W.$$

$$\text{Compression in } B = \frac{W}{2} \times \frac{\text{length of } B}{\text{length of } C}. \tag{5}$$

For girder trussed as represented in Fig. 2 under a DISTRIBUTED LOAD *W over whole girder*,

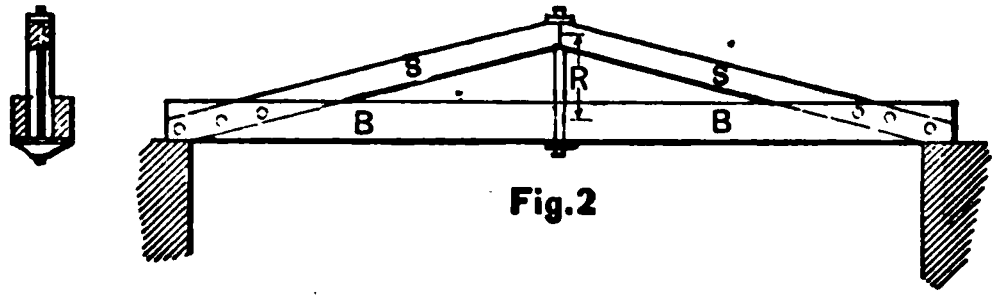


Fig. 2

$$\text{Compression in } S = \frac{3}{10} W \times \frac{\text{length of } S}{\text{length of } C}. \tag{6}$$

$$\text{Tension in } R \quad = \frac{5}{8} W.$$

$$\text{Tension in } B \quad = \frac{3}{10} W \times \frac{\text{length of } B}{\text{length of } C}. \tag{7}$$

For CONCENTRATED LOAD, *W at centre*,

$$\text{Compression in } S = \frac{W}{2} \times \frac{\text{length of } S}{\text{length of } R}. \tag{8}$$

$$\text{Tension in } R \quad = W.$$

$$\text{Tension in } B \quad = \frac{W}{2} \times \frac{\text{length of } B}{\text{length of } C}. \tag{9}$$

For double strut belly-rod truss (Fig. 3), *with* DISTRIBUTED LOAD *W over whole girder*,

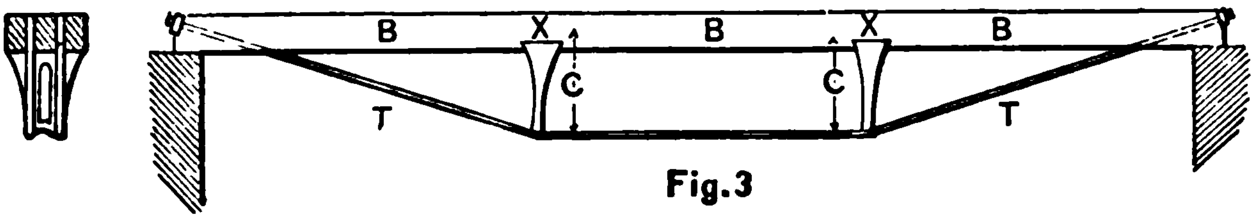


Fig. 3

$$\text{Tension in } T \quad = 0.367\,W \times \frac{\text{length of } T}{\text{length of } C}. \tag{10}$$

$$\text{Compression in } C = 0.367\,W.$$

$$\text{Comp. in } B \text{ or } D = 0.367\,W \times \frac{\text{length of } B}{\text{length of } C}. \tag{11}$$

For CONCENTRATED LOAD *W over each of the struts C,*

$$\text{Tension in } T = W \times \frac{\text{length of } T}{\text{length of } C}. \quad (12)$$

$$\text{Compression in } C = W.$$

$$\text{Comp. in } B \text{ or tension in } D = W \times \frac{\text{length of } B}{\text{length of } C}. \quad (13)$$

For girder trussed, as in Fig. 4, under a DISTRIBUTED LOAD *W over whole girder,*

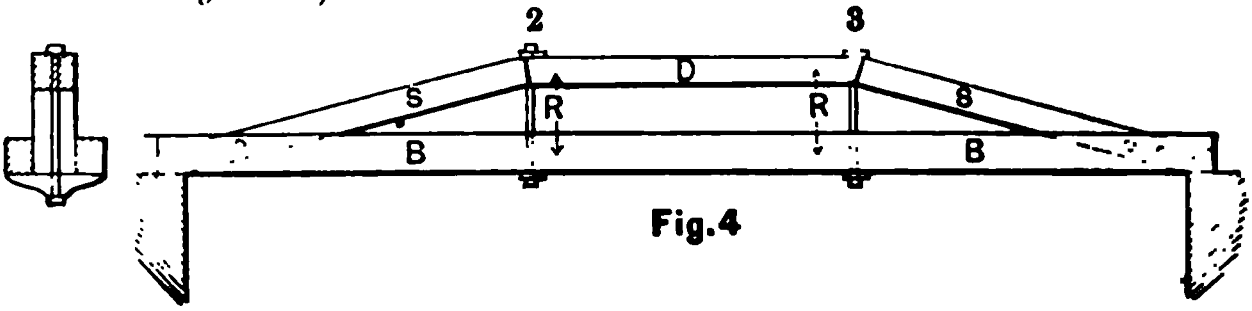


Fig.4

$$\text{Compression in } S = 0.367\,W \times \frac{\text{length of } S}{\text{length of } R}. \quad (14)$$

$$\text{Tension in } R = 0.367\,W.$$

$$\text{Tension in } B \text{ or comp. in } D = 0.367\,W \times \frac{\text{length of } B}{\text{length of } R}. \quad (15)$$

Under CONCENTRATED LOADS *W applied at 2 and 3.*

$$\text{Compression in } S = W \times \frac{\text{length of } S}{\text{length of } R}. \quad (16)$$

$$\text{Tension in } R = W.$$

$$\text{Tension in } B \text{ or comp. in } D = W \times \frac{\text{length of } B}{\text{length of } R}. \quad (17)$$

Trusses such as shown in Figs. 3 and 4 should be divided so that the rods *R*, or the struts *C*, shall divide the length of the girder into three equal or nearly equal parts. The lengths of the pieces *T*, *C*, *B*, *R*, *S*, etc., should be measured on the centres of the pieces. Thus the length of *R* should be taken from the centre of the tie-beam *B* to the centre of the strut *D*; and the length of *C* should be measured from the centre of the rod to the centre of the strut-beam *B*.

After determining the strains in the pieces by these formulas, we may compute the area of the cross-sections by the following rules: —

$$\text{Area of cross-section of strut} = \frac{\text{comp. in strut}}{C}. \quad (18)$$

$$\text{Diameter of single tie-rod}^{1} = \sqrt{\frac{\text{tension in rod}}{9425}}. \quad (19)$$

[1] Allowing 12,000 pounds safe tension per square inch in the rod.

$$\text{Diameter of each of two tie-rods} = \sqrt{\frac{\text{tension in rod}}{18850}}. \quad (20)$$

For the beam *B* we must compute its necessary area of cross-section as a tie or strut (according to which truss we use), and also the area of cross-section required to support its load acting as a beam, and give a section to the beam equal to the sum of the two sections thus obtained.

$$\left.\begin{array}{l}\text{Area of cross-section of } B \text{ to} \\ \text{resist tension or compression}\end{array}\right\} = \frac{\text{tension}}{T} \text{ or } \frac{\text{comp.}}{C}. \quad (21)$$

In trusses 1 and 2,

$$\text{Breadth of } B \text{ (as a beam)} = \frac{W \times L}{2 \times D^2 \times A}. \quad (22)$$

D = [illegible]

In trusses 3 and 4,

$$\text{Breadth of } B \text{ (as a beam)} = \frac{2 \times W \times L}{5 \times D^2 \times A}. \quad (23)$$

In these formulas,

C = 1000 pounds per square inch for hard pine and oak,
800 pounds per square inch for spruce,
700 pounds per square inch for white pine,
13,000 pounds per square inch for cast-iron.

T = 2000 pounds per square inch for hard pine,
1800 pounds per square inch for spruce,
1500 pounds per square inch for white pine,
10,000 pounds per square inch for wrought-iron.

A = 100 pounds per square inch for hard pine,
75 pounds per square inch for oak and Oregon pine,
70 pounds per square inch for spruce,
60 pounds per square inch for white pine.

EXAMPLES. — To illustrate the method of computing the dimensions of the different parts of girders of this kind, we will take two examples.

1. — *Computation for a girder such as is shown in Fig. 1*, for a span of 30 feet, the truss to be 12 feet on centres, and carrying a floor for which we should allow 100 pounds per square foot. The girder will consist of three strut-beams and two rods. We can allow the belly-rod *T* to come two feet below the beams *B*, and we will assume that the depth of the beams *B* will be 12 inches; then the length of *C* (which is measured from the centre of the beam) would be 30 inches. The length of *B* would, of course, be 15 feet, and by computation, or by scaling, we find the length of *T* to be 15 feet 2½ inches.

The total load on the girder equals the span multiplied by the distance of girders on centres, times 100 pounds $= 30 \times 12 \times 100 =$ 36000 pounds.

Then we find, from Formula 1,

$$\text{Tension in rod} = \tfrac{3}{10} \text{ of } 36000 \times \frac{182\frac{2}{5} \text{ inches}}{30 \text{ inches}} = 65664 \text{ pounds};$$

and, from Formula 20,

$$\text{Diameter of each rod} = \sqrt{\frac{65664}{18850}} = 1\tfrac{7}{8} \text{ inches, nearly.}$$

The strut-beams we will make of spruce. The compression in the two strut-beams $= \frac{3}{10}$ of $36000 \times \frac{180}{30} = 64800$ pounds, or 21600 pounds for each strut. To resist this compression would require $\frac{21600}{800}$, or 27 square inches of cross-section, which corresponds to a beam $2\frac{1}{4}$ inches by 12 inches. The load on $B = \frac{1}{2}$ of 36000, or 18000 pounds; and, as there are three beams, this gives but 6000 pounds' load on each beam. Then, from Formula 22,

$$B = \frac{6000 \times 15}{2 \times 144 \times 70} = 4.46 \text{ inches},$$

and, adding to this the $2\frac{1}{4}$ inches already obtained for compression, we have for the strut-beams three $6\frac{3}{4}$-inch by 12-inch spruce beams. The load on $C = \frac{5}{8} W$, or 22500 pounds. If we are to have a number of trusses all alike, it would be well to have a strut of cast-iron; but, if we are to build but one, we might make the strut of oak. If of cast-iron, the strut should have $\frac{22500}{13000}$, or 1.8 square inches of cross-section at its smallest section, or about 1 inch by 2 inches. If of oak, it would require a section equal to $\frac{22500}{1000}$, or $22\frac{1}{2}$ square inches, $= 4\frac{1}{2}$ inches by 5 inches, at its smallest section. Thus we have found, that for our truss we shall require three strut-beams 7 inches by 12 inches (of spruce), about 31 feet long, two belly-rods $1\frac{7}{8}$ inches diameter, and a cast-iron strut 1 inch by 2 inches at the smallest end, or else an oak strut $4\frac{1}{2}$ inches by 5 inches.

2. — It is desired to support a floor over a lecture-room forty feet wide, by means of a trussed girder; and, as the room above is to be used for electrical purposes, it is desired to have a truss with very little iron in it, and hence we use a truss such as is shown in Fig. 4. ...re the girders rest on the wall, there will be brick pilasters ...g a projection of six inches, which will make the span of the ...39 feet; and we will space the rods R R so as to divide the tie- ... into three equal spans of 13 feet each. The tie-beam will

consist of two hard-pine beams, with the struts coming between them. We will have two rods, instead of one, at R, coming down each side of the strut, and passing through an iron casting below the beams, forming supports for them. The height of truss from centre to centre of timbers we must limit to 18 inches, and we will space the trusses 8 feet on centres. Then the total floor-area supported by one girder equals 8 feet by 39 feet, equal to 312 square feet. The heaviest load to which the floor will be subjected will be the weight of students, for which 75 pounds per square foot will be ample allowance; and the weight of the floor itself will be about 25 pounds; so that the total weight of the floor and load will be 100 pounds per square foot. This makes the total weight liable to come on one girder 31,200 pounds.

Then we find, Formula 14,

$$\text{Compression in struts} = 0.367\,W \times \frac{157 \text{ ins.}}{18 \text{ ins.}} = 106800 \text{ pounds.}$$

$$\text{Tension in both tie-beams} = 0.367\,W \times \frac{156 \text{ ins.}}{18 \text{ ins.}} = 106000 \text{ pounds.}$$

$$\text{Tension in both rods } R = 0.367\,W = 11450 \text{ pounds.}$$

The timber in the truss will be hard pine, and hence we must have $\frac{106800}{1000}$, or 107 square inches, area of cross-section in the strut, which is equivalent to a 9-inch by 12-inch timber; or, as that is not a merchantable size, we will use a 10-inch by 12-inch strut. The tie-beams will each have to carry one-half of 106000, or 53000 pounds; and the area of cross-section to resist this equals $\frac{53000}{2000} =$ 27 inches, or $2\frac{1}{4}$ inches by 12 inches. The distributed load on one section of each tie-beam coming from the floor-joist equals $13 \times 8 \times 100 = 10400$ pounds; and from Formula 23 we have $B = \frac{2 \times W \times L}{5 \times D^2 \times A} = \frac{2 \times 10400 \times 13}{5 \times 144 \times 100} = 3\frac{3}{4}$ inches. Then the breadth of each tie-beam must be $3\frac{3}{4}$ inches + $2\frac{1}{4}$ inches = 6 inches: hence the tie-beams will be 6 inches by 12 inches. Each rod will have to carry 5725 pounds, and their diameter will be $\sqrt{\frac{5725}{9425}} = \frac{3}{4}$ inch, nearly.

Thus we have found, for the dimensions of the various pieces of the girder:—

Two tie-beams 6 inches by 12 inches; two rods at each joint, $\frac{3}{4}$ inch diameter; and strut-pieces 10 inches by 12 inches.

CHAPTER XX.

RIVETED PLATE-IRON GIRDERS.

WHENEVER the load upon a girder or the span is too great to admit of using an iron beam, and the use of a trussed wooden girder is impracticable, we must employ a riveted iron-plate girder. Girders of this kind are quite commonly used at the present day; as they can easily be made of any strength, and adapted to any span. They are not generally used in buildings for a greater span than sixty feet. These girders are usually made either like Fig. 1

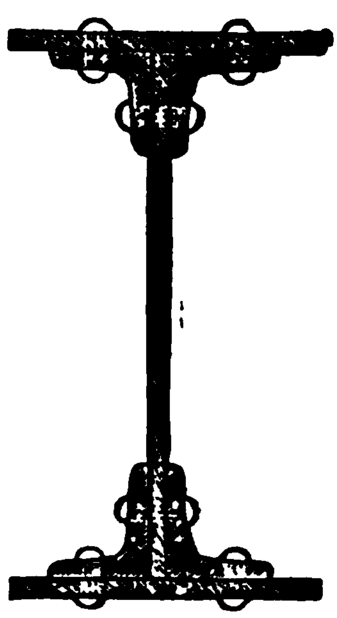


Fig. 1.

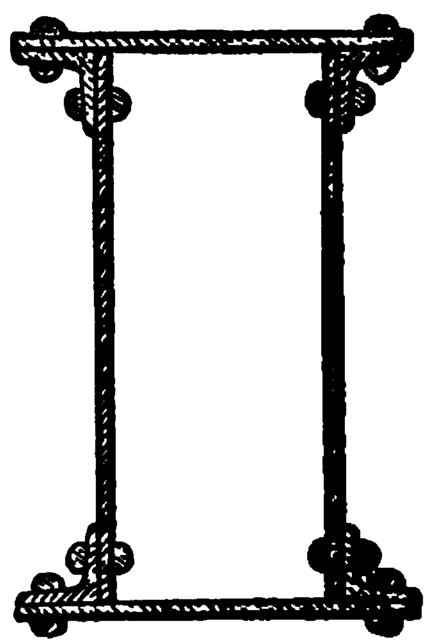


Fig. 2.

or Fig. 2, in section, with vertical stiffeners riveted to the web-plates every few feet along their length. The vertical plates, called "web-plates," are made of a single plate of wrought-iron, rarely less than one-fourth, or more than five-eighths, of an inch thick, and generally three-eighths of an inch thick. Under a distributed load, the web of three-eighths of an inch thick is generally sufficiently strong to resist the shearing-stress in the girder without ng, provided that two vertical pieces of angle-iron r veted veb, near each end of the girder. These ve es of on or T-iron, whichever is used, are c ers;" ien the girder is loaded at the centre, anu when

under a distributed load, it is necessary to use the stiffeners for the whole length of the girder, placing them a distance apart equal to the height of the girder. The web is only assumed to resist the shearing-stress in the girder. The top and bottom plates of the girder, which have to be proportioned to the loads, span, and height, are fastened to the web by means of angle-irons. It has been found, that in nearly all cases the best proportions for the angle-irons is 3 inches by 3 inches by ½ inch, which gives the sectional area of two angles five and a half square inches. The two angles and the plate taken together form the flange; the upper ones being called the "upper flange," and the lower ones the "lower flange."

RIVETS. — The rivets with which the plates and angle-irons are joined together should be three-fourths of an inch in diameter, unless the girder is light, when five-eighths of an inch may be sufficient. The spacing ought not to exceed six inches, and should be closer for heavy flanges: and in all cases it should not be more than three inches for a distance of eighteen inches or two feet from the end. Rivets should also not be spaced closer than two and a half times their diameter.

Rules for the Strength of Riveted Girders.

In calculating the strength of a riveted girder, it is customary to consider that the flanges resist the transverse strain in the girder, and that the web resists the shearing-strain. To calculate the strength of a riveted girder very accurately, we should allow for the rivet-holes in the flanges and angle-irons; but we can compute the strength of the girder with sufficient accuracy by taking the strength of the iron at ten thousand pounds per square inch, instead of twelve thousand pounds, which is used for rolled beams, and disregarding the rivet-holes. Proceeding on this consideration, we have the following rule for the strength of the girder: —

$$\text{Safe load in tons} = \frac{10 \times \text{area of one flange} \times \text{height}}{3 \times \text{span in feet}}. \qquad (1)$$

$$\left.\begin{array}{c}\text{Area of one flange}\\ \text{in square inches}\end{array}\right\} = \frac{3 \times \text{load} \times \text{span in feet}}{10 \times \text{height of web in inches}}. \qquad (2)$$

The height of the girder is measured in inches, and is the height of the web-plate, or the distance *between* the flange-plates. The web we may make either one-half or three-eighths of an inch thick; and, if the girder is loaded with a concentrated load at the centre or any other point, we should use vertical stiffeners the whole length of the girder, spaced the height of the girder apart.

If the load is distributed, divide one-fourth of the whole load on the girder, in tons, by the vertical sectional area of the web-plate: and if the quotient thus obtained exceeds the figure given in the following table, under the number nearest that which would be obtained by the following expression, $\frac{1.4 \times \text{height of girder}}{\text{thickness of web}}$, then stiffening pieces will be required up to within one-eighth of the span from the middle of the girder.

$\frac{d \times 1.4}{t}$	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100
	3.08	2.84	2.61	2.39	2.18	1.99	1.82	1.66	1.52	1.40	1.28	1.17	1.08	1.00	0.92

EXAMPLE. — A brick wall 20 feet in length, and weighing 40 tons, is to be supported by a riveted plate-girder with one web. The girder will be 24 inches high. What should be the area of each flange, and the thickness of the web?

Ans. Area of one flange $= \frac{3 \times 40 \times 20}{10 \times 24} = 10$ square inches. Subtracting 5 square inches for the area of two 3-inch by 3-inch angle-irons, we have 5 square inches as the area of the plate. If we make the plate 8 inches wide, then it should be 5 ÷ 8, or $\frac{5}{8}$ of an inch thick. The web we will make $\frac{3}{8}$ of an inch thick, and put two stiffeners at each end of the girder. To find if it will be necessary to use more stiffeners, we divide $\frac{1}{4}$ of 40 tons, equal to 10 tons, by the area of the vertical section of the web, which equals $\frac{3}{8}$ of an inch $\times$ 24 inches = 9 square inches, and we obtain 1.11. The expression $\frac{1.4 \times \text{height of girder}}{\text{thickness of web}}$, in this case, equals 89.6. The number nearest this in the table is 90, and the figure under it is 1.08, which is a little less than 1.11; showing that we must use vertical stiffeners up to within 3 feet of the centre of the girder. These vertical stiffeners we will make of $2\frac{1}{2}$-inch by $2\frac{1}{2}$-inch angle-irons. From the formula for the area of flanges, the following table has been computed, which greatly facilitates the process of finding the necessary area of flanges for any given girder.

Table of Co-efficient of Flanges for Riveted Girders.

Co-efficient for determining the area required in flanges, allowing 10,000 pounds per square inch of cross-section fibre strain : —

RULE. — Multiply the load, in tons of 2000 pounds uniformly distributed, by the co-efficient, and divide by 1000 pounds. The quotient will be the gross area, in square inches, required for each flange.

EXAMPLE. — Let us take the same girder that we have just computed. Here the span was 20 feet, and the depth of girder 24 inches. From the table we find the co-efficient to be 250, and multiplying this by the load, 40 tons, and dividing by 1000, we have 10 square inches as the area of one flange, being the same result as that obtained before.

Girders intended to carry plastering should be limited in depth (out to out of web) to one-twenty-fourth of the span-length, or half an inch per foot of span: otherwise the deflection is liable to cause the plastering to crack. In heavy girders, a saving of iron may often be made by reducing the thickness of the flanges towards the ends of the girder, where the strain is less. The bending-moment at a number of points in the length of the girder may be determined, and the area of the flange at the different points made proportional to the bending-moments at those points. The thickness of the flanges is easily varied, as required by forming them of a sufficient number of plates to give the greatest thickness, and allowing them to extend on each side of the centre, only to such distances as may be necessary to give the required thickness at each point. The deflection of girders so formed will be greater than those of uniform cross-section throughout.

TABLES OF SAFE LOADS FOR RIVETED PLATE-IRON GIRDERS.

The tables given on pp. 414 and 415 have been computed according to the formula on p. 411, to give an idea of the size of girder that will be required for a given load, of the heights and spans indicated.

If it is remembered that the strength of a girder depends directly as the area of its flanges and its height, the width and thickness of the flange plate may be changed, *provided the area remains the same*, without altering its strength. Thus a girder 36″ high, with flanges formed of 4½″ × 4½″ × ½″ angles, and ½″ × 24″ plate, would be as strong as one with the same angles and 1″ × 12″ plate, provided the web plates are properly stiffened, as described on p. 347.

In computing the weight of the girders in the tables, no allowance has been made for stiffeners. In computing the strength of riveted girders, it will be convenient to know that —

The area of two	3″ × 3″ × ½″	angle-irons	= 5.5	square inches.
"	3½″ × 3½″ × ½″	"	= 6.4	"
"	4″ × 4″ × ½″	"	= 7.4	"
"	4½″ × 4½″ × ½″	"	= 8.4	"

STRENGTH OF RIVETED WROUGHT-IRON GIRDERS.

20	41	50	62	75	46	56	70	84	61	78	92	110	20
22	38	45	57	68	42	51	64	76	56	67	84	100	22
24	35	41	52	62	39	47	58	70	51	61	77	92	24
25	33	40	50	60	37	45	56	67	49	59	74	88	25
26	32	38	48	58	36	43	54	64	47	57	71	85	26
28	30	36	44	53	33	40	50	60	44	53	66	78	28
30	28	33	41	50	31	37	47	56	41	49	61	73	30
32	26	31	39	46	29	35	44	52	38	46	57	69	32
34	24	29	37	44	27	33	41	49	36	43	54	65	34
36	23	28	34	42	26	31	39	47	34	41	51	61	36
38	22	26	33	40	24	29	37	44	32	38	48	58	38
40	20	25	31	37	23	28	35	42	30	37	46	55	40
42	–	23	30	35	–	26	33	40	–	–	44	52	42
45	–	–	28	33	–	–	31	37	–	–	–	49	45
48	–	–	–	31	–	–	–	35	–	–	–	46	48
50	–	–	–	30	–	–	–	33	–	–	–	44	50

STRENGTH OF RIVETED WROUGHT-IRON GIRDERS.

Span in Ft.	... Foot, 178 lbs.	86 lbs.	lbs.	... n. ... hes. 218 lbs.	204 lbs.	... n. ... hes. 214 lbs.	... n. ... hes. 229 lbs.	lbs.		Same Section. Height, 24 inches. Weight per Foot, 257 lbs.		lbs.	Span in Ft.
	Safe Load in Tons of 2,000 Lbs., Equally Distributed.												
20	58	70	87	105	72	86	108	130	85	102	128	154	20
24	48	58	73	87	60	72	90	108	71	85	106	128	24
26	45	54	67	81	55	66	83	100	65	79	98	118	26
28	42	50	62	75	51	61	77	93	61	73	91	110	28
30	39	47	58	70	48	57	72	86	57	68	85	102	30
32	36	44	55	66	45	54	67	81	53	64	80	96	32
34	34	41	51	62	42	51	63	76	50	60	75	90	34
36	32	39	48	58	40	48	60	72	47	57	71	85	36
38	30	37	46	55	38	45	57	68	44	54	67	81	38
40	29	35	44	52	36	43	54	65	42	51	64	77	40
42	-	33	42	50	-	41	51	62	-	49	61	73	42
44	-	32	40	48	-	39	49	59	-	47	58	70	44
46	-	-	38	46	-	-	47	56	-	-	56	67	46
48	-	-	-	44	-	-	-	54	-	-	-	64	48
50	-	-	-	42	-	-	-	52	-	-	-	61	50

Steel Beam Girders.

An economical style of box girder, well adapted for short span lengths, is one composed of a pair of I-beams with top and bottom flange plates. Such girders are commonly used for supporting interior walls in buildings.

The following tables give the safe loads for "Carnegie" beams, with different thicknesses of plates. They were prepared for *steel* girders on account of the advantages possessed by steel beams over beams of iron. The former are more economical of section and permit the use of a higher unit strain than the latter.

The values given in the tables are founded upon the moments of inertia of the various sections. Deductions were made for the rivet holes in both flanges. The maximum strain in extreme fibres was limited to 13,000 lbs. per square inch, while in the tables on rolled steel beams a fibre strain of 16,000 lbs. was used. This reduction was made in order to amply compensate for the deterioration of the metal around the rivet holes from punching.

Box girders should not be used in damp or exposed places, since the interior surfaces do not readily admit of repainting.

EXAMPLE.—A 13″ brick wall, 15 feet high, is to be built over an opening of 24 feet. What will be the section of the girder required?

Ans.—Assuming 25 feet as the distance, centre to centre of bearings, the weight of the wall will be $25 \times 15 \times 121 = 45,375$ lbs., or 22.68 tons.

On page 420 we find that a girder composed of two 12″ steel beams, each weighing 32.0 lbs. per foot, and two $14'' \times \frac{1}{2}''$ flange plates will carry safely, for a span of 25 feet, a uniformly distributed load of 23.23 tons, including its own weight. Deducting the latter, 1.42 tons, given in the next column, we find 21.81 tons for the value of the safe net load, which is 1.07 tons less than required. From the following column we find that by increasing the thickness of the flange plates $\frac{1}{16}''$ we may add 1.52 tons to the allowable load. This will more than cover the difference. Hence the required section will be two 12″ steel beams 32.0 lbs. per foot, and two $14'' \times \frac{9}{16}''$ steel cover plates.

27

STEEL BEAM GIRDERS.

SAFE LOADS IN TONS, UNIFORMLY DISTRIBUTED.

2 20″ steel (Carnegie) I-beams and 2 steel plates 16″ × ½″

Distance, centre to centre of bearings, in feet
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
36
37
38
39

Above values are based on maximum fibre strains of 15,000 lbs. per sq. in. Rivet holes in both flanges deducted. Weights of girders correspond to lengths centre to centre of bearings.

STEEL BEAM GIRDERS.

SAFE LOADS IN TONS, UNIFORMLY DISTRIBUTED.

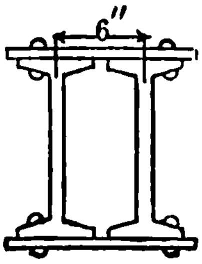
2–15″ steel (Carnegie) I-beams and 2 steel plates 14″ × [illegible]″

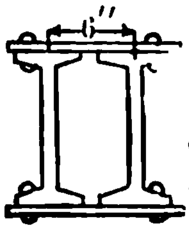
Above values are based on maximum fibre strains of 13,[illegible]00 lbs. per sq. in. Rivet holes in both flanges deducted. Weights of girders correspond to lengths, centre to centre of bearings.

STEEL BEAM GIRDERS.

SAFE LOADS IN TONS, UNIFORMLY DISTRIBUTED.

2-12″ steel (Carnegie) I-beams and 2 steel plates 14″ × ½″





Distance, centre to centre of bearings, in feet.	2 steel plates, 14″ × ½″	12″ steel I-beams, 40.0 lbs. per foot.		2 steel plates, 14″ × ½″	12″ steel I-beams, 32.0 lbs. per foot.		Increase in weight of girder for 1-16″ increase in thickness of flange plates.
	Safe load, uniformly distributed (including weight of girder), in tons of 2,000 lbs.	Weight of girder (including rivet heads), in tons of 2,000 lbs.	Increase in safe load for 1-16″ increase in thickness of flange plates.	Safe load, uniformly distributed (including weight of girder), in tons of 2,000 lbs.	Weight of girder (including rivet heads), in tons of 2,000 lbs.	Increase in safe load for 1-16″ increase in thickness of flange plates.	
10	64.94	0.65	3.75	58.08	0.57	3.81	0.03
11	59.02	0.71	3.40	52.80	0.63	3.45	0.03
12	54.12	0.78	3.12	48.40	0.68	3.17	0.03
13	49.95	0.84	2.88	44.68	0.74	2.93	0.04
14	46.39	0.91	2.68	41.48	0.80	2.72	0.04
15	43.29	0.97	2.50	38.72	0.85	2.53	0.04
16	40.59	1.04	2.34	36.30	0.91	2.38	0.05
17	38.20	1.10	2.21	34.16	0.97	2.24	0.05
18	36.08	1.17	2.08	32.27	1.03	2.11	0.05
19	34.18	1.23	1.97	30.57	1.08	2.00	0.05
20	32.47	1.30	1.87	29.04	1.14	1.90	0.06
21	30.93	1.36	1.78	27.66	1.20	1.81	0.06
22	29.52	1.43	1.70	26.40	1.25	1.73	0.06
23	28.23	1.49	1.63	25.25	1.31	1.65	0.07
24	27.06	1.56	1.56	24.20	1.37	1.58	0.07
25	25.98	1.62	1.50	23.23	1.42	1.52	0.07
26	24.98	1.69	1.44	22.34	1.48	1.46	0.08
27	24.05	1.75	1.38	21.51	1.54	1.41	0.08
28	23.19	1.82	1.34	20.74	1.60	1.36	0.08
29	22.39	1.88	1.29	20.03	1.65	1.31	0.08
30	21.65	1.95	1.25	19.36	1.71	1.27	0.09
31	20.95	2.01	1.21	18.73	1.77	1.23	0.09
32	20.29	2.08	1.17	18.15	1.82	1.19	0.09
33	19.68	2.14	1.14	17.60	1.88	1.15	0.10
34	19.10	2.21	1.10	17.08	1.94	1.12	0.10
35	18.55	2.27	1.07	16.59	1.99	1.09	0.10
36	18.04	2.34	1.04	16.13	2.05	1.06	0.10
37	17.55	2.40	1.01	15.70	2.11	1.03	0.11
38	17.09	2.47	0.99	15.28	2.17	1.00	0.11
39	16.65	2.53	0.96	14.89	2.22	0.98	0.11

Above values are based on maximum fibre strains of 13,000 lbs. per sq. in. Rivet holes in both flanges deducted. Weights of girders correspond to lengths centre to centre of bearings.

STEEL BEAM GIRDERS.

SAFE LOADS IN TONS, UNIFORMLY DISTRIBUTED.

2-10″ steel (Carnegie) I-beams and 2 steel plates 12″ × ½″

Above values are based on maximum fibre strains of 13,000 lbs. per sq. in. Rivet holes in both flanges deducted. Weights of girders correspond to lengths, centre to centre of bearings.

CHAPTER XXI.

STRENGTH OF CAST-IRON ARCH-GIRDERS, WITH WROUGHT-IRON TENSION-RODS.

Cast-iron arch-girders are now quite extensively employed to support the front or rear walls of brick buildings. Fig. 1 shows the usual form of such a girder, the section of the casting and rod being shown in Fig. 2.

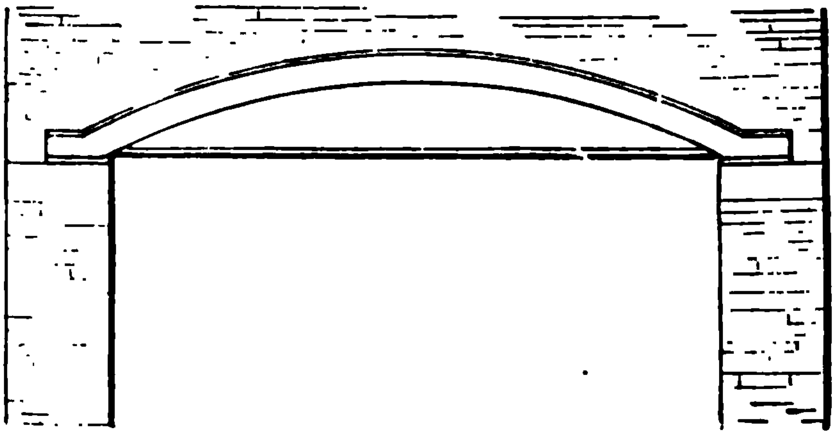


Fig. 1.

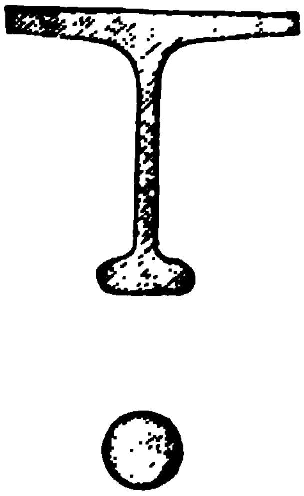


Fig. 2.

The casting is made in one piece with box ends, the latter having grooves and seats to receive the wrought-iron tie-rod.

The tie-rod is made from one-eighth to three-eighths of an inch shorter than the casting, and has square ends forming shoulders so as to fit into the castings. The rod has usually one weld on its length, and great care should be taken that this weld be perfect.

The rod is expanded by heat, and then placed in position in the casting, and allowed to contract in cooling; thus tying the two ends of the casting together to form abutments for receiving the horizontal thrust of the arch. If the rod is too long, it will not receive the full proportion of the strain until the cast-iron has so far deflected, that its lower edge is subjected to a severe tensile strength, which cast-iron can feebly resist. If the tie-rod is made too short, the casting is cambered up, and a severe initial strain put upon both the cast and wrought iron, which enfeebles both for carrying

a load. The girders should have a rise of about two feet six inches on a length of twenty-five feet.[1]

Rules for Calculating Dimensions of Girder and Rod.

A cast-iron arch-girder is considered as a long column, subject to a certain amount of bending-strain ; and the resistance will be governed by the laws affecting the strength of beams, as well as by those relating to the strength of columns.

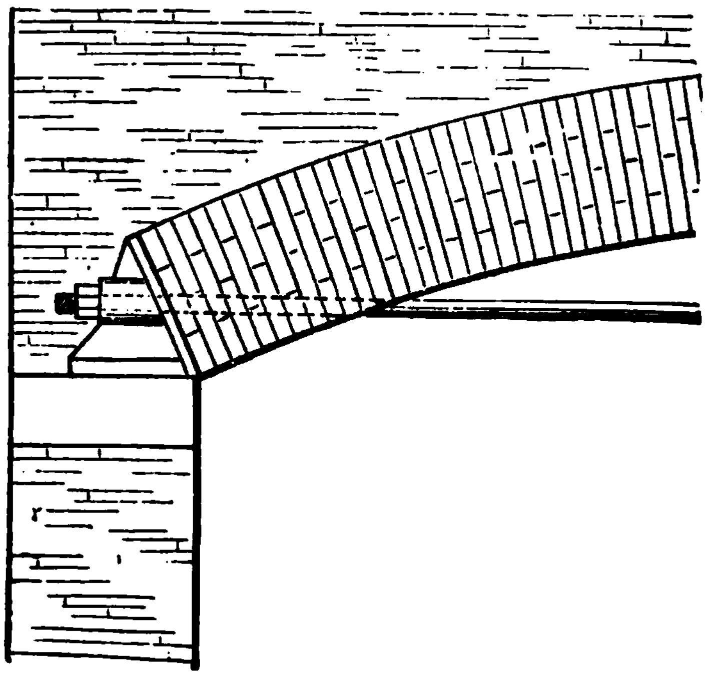


Fig. 3.

If we regard the arch as flexible, or as possessing no inherent stiffness, and the rod as a chord without weight, we can deduce the following formula for the horizontal thrust or strain: —

$$\text{Hor. thrust or strain} = \frac{\text{load per foot of span} \times \text{span in feet, squared}}{8 \times \text{rise of girder in feet}}. \quad (1)$$

From this rule we can calculate the required diameter of the tension-rod, which may be expressed thus : —

$$\text{Diameter in inches} = \sqrt{\frac{\text{load on girder} \times \text{span in feet}}{8 \times \text{rise in feet} \times 7854}}. \quad (2)$$

The rule generally used, however, in proportioning the wrought-iron tie to the cast-iron arch is *to allow one square inch of cross-section of tie-rod for every ten net tons of load imposed upon* the span of the arch.

The following table, taken from Mr. Fryer's book on "Architec-

[1] Architectural Iron-Work for Buildings. — WILLIAM J. FRYER, Jun. Pp. 38.

tural Iron-Work," shows *the section of the cast-iron arch required to support solid brick walls, and having a span of from 13 to 26 feet.*

Height of wall.	Thickness of wall.	Dimensions of Section.		
		Top flange.	Centre web.	Bulb.
40 feet.	12 inches.	12″ × 1″	12″ × $\frac{3}{4}$″	3″ × 2″
50 "	12 "	12″ × $1\frac{1}{4}$″	12″ × $\frac{7}{8}$″	3″ × 2″
40 "	16 "	12″ × $1\frac{1}{4}$″	12″ × $\frac{7}{8}$″	$3\frac{1}{2}$″ × 2″
50 "	16 "	16″ × $1\frac{1}{4}$″	12″ × 1″	4″ × 2″

Substitute for Cast-iron Arch-Girder.

In the cast-iron arch-girder with wrought-iron tension-rod, the casting only serves to resist compression. Its place can as well be filled by a brick arch footed on a pair of cast-iron skewbacks, which are themselves held in position by a pair of tie-rods, as in Fig. 3.

In this case, Formula 1 will still give the horizontal pull to be resisted by the tie-rods; but, as we must have two rods instead of one, the diameter of each will be obtained by the formula,

$$\text{Diameter of each rod in inches} = \sqrt{\frac{\text{total load on arch} \times \text{span}}{16 \times \text{rise of arch in feet} \times .7854}}. \qquad (3)$$

N.B. — The rise is measured from the centre of the rod to the centre of the arch. It will also be remembered that the span is to be *always* taken in feet, unless otherwise specified.

Example 1. — It is desired to support a 12-inch brick wall 40 feet high over an opening 20 feet wide, with a cast-iron arch-girder. What should be the dimensions of the girder?

For the casting, we find from the table that the cross-section of the flange should be 12 inches by 1 inch; of the web, 12 inches by $\frac{3}{4}$ inch; and of the bulb, 3 inches by 2 inches. We will make the rise of the girder 2 feet and 6 inches, and from Formula 2 we find[1]

$$\text{Diam. of rod in inches} = \sqrt{\frac{\text{weight of wall} \times \text{span}}{8 \times \text{rise of arch in feet} \times .7854}}$$

$$\sqrt{\frac{(20 \times 20 \times 112) \times 20}{8 \times 2\frac{1}{2} \times .7854}} = \sqrt{5.7} = 2\frac{3}{8} \text{ ins.}$$

[1] Considering that the girder would only support about twenty feet of the wall in height, the wall above that supporting itself.

CHAPTER XXII

STRENGTH AND STIFFNESS OF WOODEN FLOORS

Strength of Floors. — In calculating the strength of floor-beams, the first thing to be decided is the span of the beams, which is generally determined by the size of the opening to be covered; and the second is the load which is to come upon the floor. Wooden floor-beams should not have a span of more than twenty-five feet (if it can be so arranged); for, if they are of a greater length than this, it is difficult to stiffen them sufficiently to prevent vibration under a heavy or moving load When the distance between the bearing-walls of a building is greater than the above limit, partition-walls should be built, or else the beams should be supported by iron or wooden girders resting upon iron or wooden columns.

The Building Laws of the cities of New York and Boston require that in all buildings more than thirty feet in width, except churches, theatres, schoolhouses, car-stables, and other public buildings, the space between any two of the bearing-walls shall not be over twenty-five feet, unless girders are substituted in place of the partition-wall. Floor-beams, when supported at three or more points, should always be made continuous if possible, as the strength of each portion of the beam is thereby greatly increased.

Superimposed Loads. — There is some difference of opinion among authorities as to what should be allowed for the superimposed load upon the floor of a dwelling or upon the floors of public buildings. The New-York Building Law requires that in all buildings every floor shall have sufficient strength to bear safely upon every superficial foot of its surface seventy-five pounds, and, if used as a place of public assembly, one hundred and twenty pounds.

In dwelling-houses, where the maximum load consists of nothing but ordinary furniture and the weight of some ten or twelve people, it is not necessary to allow more than forty pounds per square foot for the superficial load; and, in most cases, eighty pounds per square foot is ample allowance for the weight of an assemblage of people. Only in cases where people are liable to be jammed together during

a panic or some unusual circumstance, is it possible to get a weight on the floor of one hundred and twenty pounds per square foot. The following table gives the weight per square foot which should be assumed, in addition to the weight of the floor, for these various cases: —

For street bridges for general public traffic,	80 lbs. per square foot.
For floors of dwellings	40 lbs. per square foot.
For churches, theatres, and ball-rooms,	80 to 120 lbs. per square foot.
For schools	80 lbs. per square foot.
For hay-lofts	80 lbs. per square foot.
For storage of grain	100 lbs. per square foot.
For warehouses and general merchandise,	250 lbs. per square foot.
For factories	100 to 400 lbs. per square foot.
For office buildings	100 lbs. per square foot.

Warehouse-floors are sometimes very heavily loaded, and for these a special computation should be made in each case.

The following table, compiled by Mr. C. J. H. Woodbury,[1] gives the floor areas, cubic space, and weights of merchandise, as usually stored in warehouses. If the goods are piled two or more cases high, the weight per square foot of floor will of course be increased in proportion. "The measurements were always taken to the outside of case or package, and gross weights of such packages are given."

MATERIAL.	MEASUREMENTS.		WEIGHTS.		
	Floor space.	Cubic feet.	Gross.	Per sq. ft.	Per cubic ft.
Wool.					
Bale East India	3.0	12.	340	113	28
" Australia	5.8	26.	385	66	15
" South America . . .	7.0	34.	1000	143	29
" Oregon	6.9	33.	482	70	15
" California	7.5	33.	550	73	17
Bag Wool	5.0	30.	200	40	7
Sacks of Scoured Wool . .	–	–	–	–	5
Woollen Goods.					
Case Flannels	5.5	12.7	220	40	17
" Flannels, heavy . . .	7.1	15.2	330	46	22
" Dress Goods	5.5	22.0	460	84	21
" Cassimeres	10.5	28.0	550	52	20
" Underwear	7.3	21.0	350	48	16
" Beavers	10.3	35.0	450	44	13
" Horse Blankets . . .	4.0	14.0	250	63	18
Cotton, etc.					
Bale	8.1	44.2	515	64	12
" Compressed	4.1	21.6	550	134	25
" [illegible] Compressed .	1.25	3.13	125	100	40
" Jute	2.4	9.9	300	125	30
" Jute Lashings . . .	2.6	10.5	450	172	43
" Manila	3.2	10.9	280	88	26
" Hemp	8.7	34.7	700	81	20
" Sisal	5.3	17.0	400	75	24

[1] The Fire Protection of Mills, p. 118.

Material.	Measurements.		Weights.		
	Floor space.	Cubic feet.	Gross.	Per sq. ft.	Per cubic ft.
Goods.					
leached Jeans	4.0	12.5	300	72	24
k	1.1	2.3	75	68	33
vn Sheetings	3.6	10.1	235	65	23
ched Sheetings	4.8	11.4	330	69	30
ts	7.2	19.0	295	41	16
t Cloth	4.0	9.3	175	44	19
ts	4.5	13.4	420	93	31
ings	3.3	8.8	325	99	37
tton Yarn	–	–	–	–	11
	–	–	130	–	30
ging	1.4	5.3	100	70	24
in Bales.					
nen	8.5	39.5	910	107	23
tton	9.2	40.0	715	78	18
tton	7.6	30.0	442	59	15
avings	7.5	34.0	507	68	15
	16.0	65.0	450	28	7
	7.5	30.0	600	80	20
s	2.8	11.1	400	143	36
d Book	–	–	–	–	50
endered Book	–	–	–	–	69
er	–	–	–	–	38
ard	–	–	–	–	33
Board	–	–	–	–	59
	–	–	–	–	64
g	–	–	–	–	10
	–	–	–	–	37
Bags	4.2	4.2	165	39	39
Bulk	–	–	–	–	44
"	–	–	–	–	39
" mean	–	–	–	–	41
lour on side	4.1	5.4	218	53	40
" on end	3.1	7.1	218	70	31
Bags	3.6	3.6	112	31	31
in Barrels	3.7	5.9	218	59	37
ags	3.3	3.6	96	29	27
lay	5.0	20.0	284	57	14
erick Compressed	1.75	5.25	125	72	24
" "	1.75	5.25	100	57	19
" "	1.75	5.25	150	86	29
" "	1.75	5.25	100	57	19
tuffs, etc.					
l Bleaching Powder,	11.8	39.2	1200	102	31
Soda Ash	10.8	29.2	1800	167	62
go	3.0	9.0	385	128	43
h	4.0	3.3	150	38	45
ac	1.6	4.1	160	100	39
oda in iron drum	4.3	6.8	600	140	88
arch	3.0	10.5	250	83	23
earl Alum	3.0	10.5	350	117	33
act Logwood	1.06	.8	55	52	70
ime	3.6	4.5	225	63	50
ement, American	3.8	5.5	325	86	59
" English	3.8	5.5	400	105	73
aster	3.7	6.1	325	88	53

MATERIAL.	MEASUREMENTS.		WEIGHTS.		
	Floor space.	Cubic feet.	Gross.	Per sq. ft.	Per cubic ft.
Dye Stuffs, etc.—*Cont'd.*					
Barrel Rosin	3.0	9.0	430	143	48
" Lard Oil	4.3	12.3	422	98	34
Rope	-	-	-	-	42
Miscellaneous.					
Box Tin	2.7	0.5	139	99	278
" Glass	-	-	-	-	60
Crate Crockery	9.9	39.6	1600	162	40
Cask Crockery	13.4	42.5	600	52	14
Bale Leather	7.3	12.2	190	26	16
" Goatskins	11.2	16.7	300	27	18
" Raw Hides	6.0	30.0	400	67	13
" " " compressed,	6.0	30.0	700	117	23
" Sole Leather	12.6	8.9	200	22	16
Pile Sole Leather	-	-	-	-	17
Barrel Granulated Sugar. .	3.0	7.5	317	106	42
" Brown Sugar . . .	3.0	7.5	340	113	45
Cheese	-	-	-	-	30

Weight of the Floor itself.—Having decided upon the span of the floor beams and upon the superimposed load, we must next consider the weight of the floor itself.

Wooden floors in dwellings weigh, on the average, from seventeen to twenty two pounds per square foot of floor, including the weight of the plastering on the under side. For ordinary spans the weight may be taken at twenty pounds per square foot, and, for long spans, twenty two pounds per square foot. For floors in public buildings, the weight per square foot seldom exceeds twenty-five pounds, and it may safely be assumed at that amount.

In warehouse floors, which have to sustain very heavy loads, the weight per square foot may sometimes be as great as forty or fifty pounds; and in such cases the approximate weight of the floor per square foot should be first calculated.

Factor of Safety to be used.—In considering the load on a floor, it should be remembered that the effect of a load suddenly applied upon a beam is twice as great as that of the same load gradually applied; and hence the factor of safety used for the former should be twice as great as that for the latter. The load caused by a crowd of people is usually considered to produce an effect which is a mean between that of the same load when gradually and when suddenly applied; and hence a factor of safety is employed which is a mean between that for a live and for a dead load.

The factors of safety for floor-timbers adopted by the best engineers vary from 3 to 5. For short spans in ordinary dwellings, public buildings, and stores, 3 is probably amply sufficient for

strength; but for long spans, and floors in factories and machine-shops, a factor of safety of 5 should often be used.[1]

Rules for the Strength of Floor-beams.—In considering the strength of a floor, we assume it to be equally loaded over its whole surface, as this would be the severest strain to which the timbers could be subjected. Hence, in calculating the dimensions of the floor-beams, we use the formula for a distributed load. That formula is, for rectangular beams,

$$\text{Safe load} = \frac{2 \times \text{breadth} \times \text{depth squared} \times A}{\text{span in feet} \times S} \qquad (1)$$

S being the factor of safety.

For floor-beams the safe load is represented by the superimposed load and weight of floor supported by each beam.

The area of floor supported by each beam equals the length of beam multiplied by the distance between centres. If we let f denote the weight of the superimposed load per square foot of floor surface, and f' the weight of one square foot of the floor itself, then the total weight per square foot will be $(f + f')$ pounds, and the total load on each beam will equal

$$\text{Length of beam} \times \text{distance between centres} \times (f + f').$$

Now, if we substitute this expression in place of the safe load in the above formula, and solve for the depth, we shall have,

$$\text{Square of depth.} = \frac{S \times \text{dist. bet. centres} \times \text{length squared} \times (f + f')}{2 \times \text{breadth} \times A}; \qquad (2)$$

or, if we solve for the distance between centres, we shall have,

$$\text{Distance between centres in feet} = \frac{2 \times \text{breadth} \times \text{depth squared} \times A}{S \times \text{length squared} \times (f + f')}. \qquad (3)$$

N. B.—The length and distance between centres must be taken in *feet*, and the length means only the distance between supports, or the clear span.

The values of the constant A for the four woods in general use are as follows:

Spruce	210	Oak	225
Hard pine	300	White pine	180

Formulas 2 and 3 apply to all floors supported by rectangular beams, whatever be the factor of safety employed, the weight of

[1] Until very recently it has been our custom to use factors of safety twice as great as these; but, as we have had occasion to reduce the constants for strength to about one-half of that formerly used, we have reduced the factors of safety accordingly. It will be found that the result is the same as that obtained by the rules of other writers.

the superimposed load, or of the floor itself. To illustrate the application of these formulas, we will give two examples such as are constantly occurring in practice.

EXAMPLE 1.—What should be the dimensions of the spruce floor beams in a dwelling, the beams to have a span of 15 feet, and to be placed 16 inches, or $1\frac{1}{3}$ feet, on centres?

Ans. In this case we would use a factor of safety of 4: f should be taken at 40 pounds, f' at 2[illegible] pounds, and A is 210 pounds. Assume 2 inches for the breadth. Then, by Formula 2,

$$\text{Square of depth} = \frac{4 \times 1\frac{1}{3} \times 225 \times 60}{2 \times 2 \times 210} = 85.5$$

The depth $= \sqrt{85.5}$ = a little over 9 inches. Hence, to have the requisite strength, the beams should be 2×10 inches.

EXAMPLE 2.—It is desired to use 2 by 10 inch yellow-pine beams in the floor of a church, the beams to have a span of 16 feet. What distance should they be spaced on centres?

Ans. Here $S = 5$, $f = 100$ pounds, $f' = 25$ pounds, and $A = 300$ pounds. Then, by Formula 3, we have,

$$\text{Distance between centres} = \frac{2 \times 2 \times 100 \times 300}{5 \times 256 \times 125} = 0.75 \text{ ft., or 9 ins.}$$

Hence the floor will be sufficiently strong if the beams are placed 9 inches on centres.

Bridging of Floor-beams.—By "bridging" is meant a system of bracing floor-beams, either by means of small struts, as in Fig. 1, or by means of single pieces of boards at right angles to the joists, and fitting in between them.

The effect of this bracing is decidedly beneficial in sustaining any *concentrated* weight upon a floor; but it does not materially strengthen a floor to resist a uniformly distributed load. The bridging also stiffens the joists, and prevents them from turning sidewise. It is customary to insert rows of cross-bridging at from every five to eight feet in the

Fig. 1.

length of the beams, and to be effective they should be in straight lines along the floor, so that each strut may abut directly upon

those adjacent to it. The method of bridging shown in Fig. 1, and known as "cross-bridging," is considered to be by far the best, as it allows the thrust to act parallel to the axis of the strut, and not across the grain, as must be the case where single pieces of board are used.

The bridging should be of $1\frac{1}{4}$-inch by 3-inch stock.

Carriage-beams, Headers, and Tail-beams.—Fig. 2 represents the plan of the timbers of a floor, having a stairway opening on each side. The short beams, as KL, are called the "tail-beams:" the beams EF and GH, which support the tail-beams, are called the "headers:" and the beams AB and CD, the "carriage-beams," or "trimmers."

The *tail-beams* are calculated in the same way as ordinary floor-joist; but it is evident that the headers and trimmers will require separate computations.

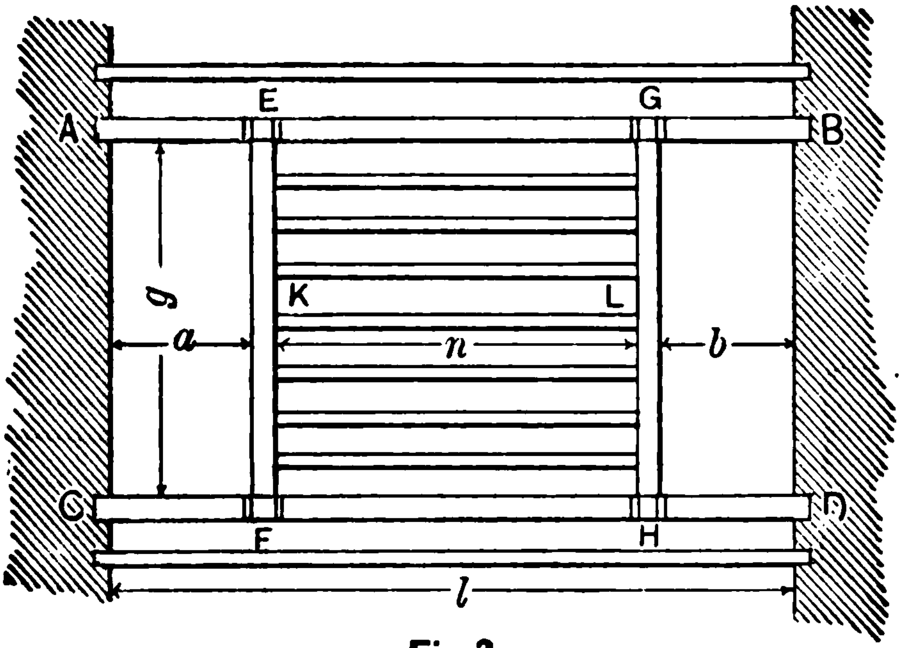


Fig. 2

It would be very difficult to give formulas that would serve for every case of trimmers and headers; and the best way in any case is to find the load which the trimmer has to carry, and then, from the formulas already given, determine the required dimensions. In a floor such as is represented in Fig. 2, it is evident that the floor-area supported by EF or $GH = g \times \frac{1}{2}n$. Multiplying this area by $(f + f')$, we should have the load which each header would be required to support; and then, by Formula 9, Chap. XV., we could determine its necessary dimensions.

As the headers are weakened by the tail-beams being mortised into them, a certain allowance should be made for mortising in calculating the dimensions. In ordinary cases it would probably be enough to make the breadth from one to two inches more than the calculated dimensions.

The trimmers, *AB* and *CD*, have to support one-half of the load carried by *EF* plus one-half the load carried by *GH*, and also one-half of the load supported by the ordinary joist. The best way in which to calculate such a trimmer is to consider it to be made up of two beams placed side by side, one to carry the end of the headers *EF* and *GH*, and the second being one-half the thickness of the ordinary joist. The breadth of the part carrying the ends of the trimmers could then be calculated by Formula 13, Chap. XV., and the total breadth of the trimmers found by adding together the breadths of the two parts into which it is supposed to be divided. We have not the space here to consider further the strength of headers and trimmers, but would refer any readers desiring further information on the subject to Hatfield's "Transverse Strains," where they will find the subject fully discussed.

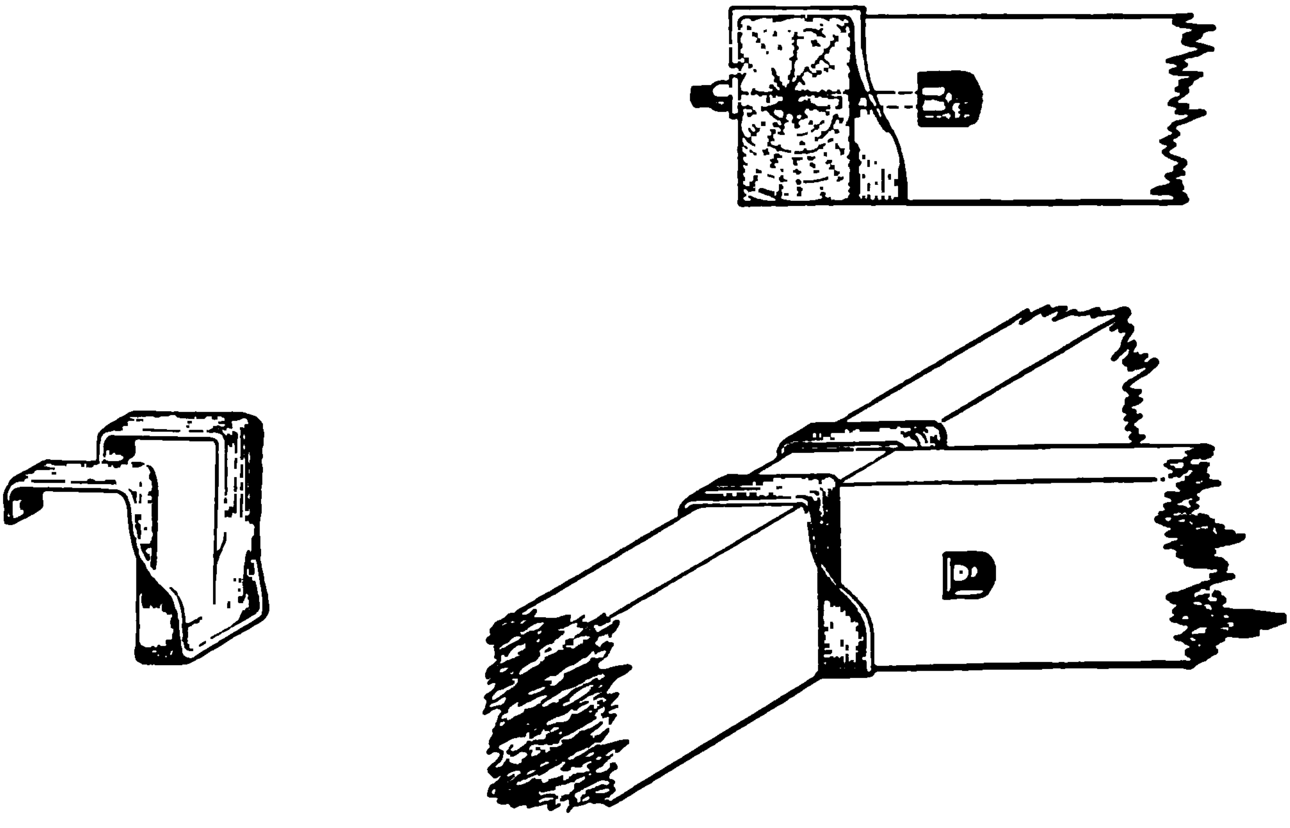


Fig. 3

Stirrup-Irons. — At the point of connection of the end of the header with the trimmer, the load on the trimmer coming from the header is a concentrated one; and all mortising at this point, to receive the header, should be avoided. It is now the custom, in first-class construction, to support the ends of headers by means of stirrup-irons, as shown in Fig. 3. The Boston and New-York Building Laws require that "every trimmer or header more than four feet long, used in any building except a dwelling, shall be hung in stirrup-irons of suitable thickness for the size of the timbers."

It is evident that each vertical part of the stirrup will have to

carry one-fourth of the load on the header; and we can easily deduce the rule,

$$\text{Area of cross-section of stirrup} = \frac{\text{load borne by header}}{40000}. \quad (4)$$

The stirrup-irons are generally made of iron bars about two inches wide and three-eighths or one-half inch thick.

The headers are also generally bolted to the trimmer, as shown in the same figure; so that the trimmers shall not spread, and let the headers fall.

Joist Hangers.—On page 437*f* are shown two styles of patented joist hangers, which are intended to take the place of the stirrup iron, at less cost.

Girders. — Formulas 2 and 3 will also apply to wooden girders supporting the floor-joist, neglecting the weight of the girder itself. In this case the distance between centres would, of course, mean the distance between the centres of the girders. The application of these formulas to girders being the same as for the floor-joist, it seems hardly necessary to illustrate by examples.

Solid or Mill Floors.

By *Solid or Mill Floors* we mean a floor constructed of large beams spaced about eight feet on centres, and covered with plank of suitable thickness, and this, again, covered with maple or hard-pine flooring as desired. Such floors will be found fully described in Chap. XXIV.

For calculating the large timbers, the best method is to compute the greatest load that the beam is ever liable to carry, and then determine the necessary size of timber by means of the proper formula, which may be found in Chap. XV.; or if the beams are spaced a regular distance apart, and have only a uniformly distributed load to carry, they may be computed by Formulas 2 and 3, given above.

The floor-plank may be computed for their *strength* by the following formula, supposing the load to be uniformly distributed:—

$$\text{Thickness of plank} = \sqrt{\frac{\text{weight per square foot} \times L^2 \times S}{24 \times A}}. \quad (5)$$

They would, however, bend too much, when proportioned by this formula, for use in mills, and in buildings where the under side of the plank must be plastered.

For such buildings the thickness of the plank should be proportioned by the formula for stiffness, which is,

$$\text{Thickness of plank} = \sqrt[3]{\frac{\text{weight per square foot} \times L^3}{19.2 \times e}} \qquad (6)$$

e being the constant for deflection given in Chap. XVI.

For spruce, $e = 100$ pounds, and for hard pine 137 pounds, for a deflection of one-thirtieth of an inch per foot of span.

The weight per square foot should include the superficial load on the floor and the weight of the plank and upper flooring.

EXAMPLE. — What should be the thickness of the spruce plank in a mill where the beams are spaced 8 feet on centres, and the superficial load may attain 120 pounds per square foot?

Ans. The weight of the plank and flooring, with deafening between, will weigh about 15 pounds per square foot, making the total load per square foot 135 pounds. Then, from Formula 6,

$$\text{Thickness of plank} = \sqrt[3]{\frac{135 \times 8 \times 8 \times 8}{19.2 \times 100}} = \sqrt[3]{36} = 3.3 \text{ inches, or } 3\tfrac{1}{2}\text{-inch plank.}$$

The plank would probably come in two or three lengths, which would make the floor considerably stiffer; but, as there might occur cases when the floor would have to sustain heavy concentrated loads for a short time, it would be hardly wise to use a less thickness of plank.

The following table, taken from Mr. C. J. H. Woodbury's excellent work on "The Fire Protection of Mills, and Construction of Mill-Floors," shows the dimensions of beams, and thickness of plank for warehouse-floors loaded with from fifty to three hundred pounds per square foot, the beams being spaced eight feet on centres. The plank is supposed to be of spruce, and the beams of hard or Southern pine.

Several sizes of beams are given; so that a selection of those which will apply most conveniently to any specific case may be made.

STRENGTH OF SOLID TIMBER AND PLANK FLOORS.

(By C. J. H. Woodbury.)

Weight per Square Foot of Floor.				Dimensions of Beams.			Thickness of floor-plank, in inches.
Superficial load.	Weight of beam, in lbs.	Weight of floor-plank.	Total.	Depth, in inches.	Breadth in inches.	Span, in feet.	
50	3.00	6.07	59.07	12	6	20.95	2.43
	4.08		60.15	14	7	26.16	
	5.33		61.40	16	8	31.63	
75	3.00	7.40	85.40	12	6	17.42	2.96
	4.08		86.48	14	7	21.82	
	5.33		87.73	16	8	26.46	
100	3.00	8.55	111.55	12	6	15.25	3.42
	4.08		112.63	14	7	19.12	
	5.33		113.88	16	8	23.23	
125	3.00	9.55	137.55	12	6	13.73	3.82
	4.08		138.63	14	7	17.23	
	5.33		139.88	16	8	20.96	
150	3.00	10.45	163.45	12	6	12.59	4.18
	4.08		164.53	14	7	15.82	
	5.33		165.78	16	8	19.25	
175	3.00	11.26	189.26	12	6	11.71	4.51
	4.08		190.34	14	7	14.70	
	5.33		191.59	16	8	17.91	
200	3.00	12.05	215.05	12	6	10.98	4.82
	4.08		216.13	14	7	13.80	
	5.33		217.38	16	8	16.81	
225	3.00	12.75	240.75	12	6	10.38	5.11
	4.08		241.83	14	7	13.06	
	5.33		243.08	16	8	15.90	
250	3.00	13.45	266.45	12	6	9.86	5.38
	4.08		267.53	14	7	12.40	
	5.33		268.78	16	8	15.08	
275	3.00	13.55	291.55	12	6	9.43	5.62
	4.08		292.63	14	7	11.86	
	5.33		293.88	16	8	14.46	
300	3.00	14.72	317.72	12	6	9.03	5.89
	4.08		318.80	14	7	11.36	
	5.33		320.05	16	8	13.85	

Stiffness of Wooden Floors.

Floors in first-class buildings should possess something more than mere strength to resist fracture: they should have sufficient stiffness to prevent the floor from bending, under any load, enough to cause the ceiling to crack, or to present a bad appearance to the eye. To obtain this desired quality in floors, it is necessary to calculate the requisite dimensions of the beams by the formulas for stiffness; and, if the dimensions obtained are larger than those

obtained by the formulas for strength, they should be adopted, instead of those obtained by the latter formulas. The only way in which we can be sure that a beam is both strong enough and stiff enough to bear a given load is to calculate the required dimensions by both the formula for strength and the formula for stiffness, and take the larger dimensions obtained. As a general rule, those beams in which the proportion of *depth* to *length* is very *small* should be calculated by the formulas for *strength*, and *vice versa*. Formula 10, Chap. XVI., gives the load which a given beam will carry without deflecting more than one-fortieth or one-thirtieth of an inch per foot of span, according to the value of e which we use. Formula 11, Chap. XVI., gives the dimensions of the beam to carry a given load under the same conditions.

In the case of floor-beams, the load is given, and is represented, as we saw under the *Strength of Floors*, by the expression,

$$\text{Distance between centres in feet} \times \text{length in feet} \times (f + f').$$

Then, if we substitute this expression in place of the load in Formula 11, Chap. XVI., we shall have the formula,

$$\text{Breadth} = \frac{5 \times \text{dist. between centres} \times \text{cube of length} \times (f + f')}{8 \times \text{cube of depth} \times e} \qquad (7)$$

or

$$\text{Dist. between centres} = \frac{8 \times \text{breadth} \times \text{cube of depth} \times e}{5 \times \text{cube of length} \times (f + f')}. \qquad (8)$$

The proper values for f and f' have been given under the *Strength* of floors in the preceding part of this chapter, and the value of e for any given case may be found in Chap. XVI.[1]

In ordinary floors, when the values of f used are those recommended above, a deflection of one-thirtieth of an inch per foot of span may safely be allowed, as the floors would probably be very rarely loaded to their utmost capacity, and then but for a short time: so that it would have no injurious effects.

As an example showing the application of Formula 7, we will take Example 1 under the strength of wooden floors.

In this example, the beams were to have a span of 15 feet, and be placed 1½ feet on centres: f was taken at 40 pounds, and f' at 20 pounds. What should be the dimensions of the beams, that they may safely carry the load upon them without deflecting more than $\frac{1}{30}$ of an inch per foot of span?

[1] The values for e, for spruce, hard pine, and oak, are,

	Def. $= \frac{1}{360} L$.	Def. $= \frac{1}{480} L$
Spruce	100	75
Hard pine	137	103
Oak	95	72

Ans. We have simply to substitute our known quantities in Formula 7, assuming the depth at 10 inches, and taking the value of e at 100 pounds, the beams being of spruce.

Performing the operation, we have,

$$\text{Breadth} = \frac{5 \times 1\frac{1}{3} \times 15^3 \times (40 + 20)}{8 \times 10^3 \times 100} = 1.68 \text{ inches.}$$

This gives us about the same dimensions that we obtained when considering the beam in regard to its strength only: hence a beam two by ten inches would fulfil both the conditions of strength and stiffness.

In the case of headers, stringers, etc., where the joist has to carry not only a distributed load, but also one or more concentrated loads applied at different points of the beam, the required dimensions can best be obtained by considering the beam to be made up of a number of pieces of the same depth, placed side by side, and computing the required breadth of beams of that depth to carry each of the loads singly, and then taking the sum of the breadths for the breadth required.

The formula for stiffness of *plank-floors* has already been given on p. 434.

Dimensions of Joists and Girders for Different Loads and Spans.

To enable an architect to tell at a glance the size of joists and girders required for the ordinary classes of buildings, the author has computed the following tables, which give the dimensions required for spans from 10 to 24 feet for joists, and also the maximum distance that the joists should be spaced on centers. Dimensions for girders are given for different spans and spacings.

The beams and girders in the first three classes were computed from the tables on pages 388, 389, and 390, and in class D from the tables on pages 377 and 379.

The application of the tables will doubtless be evident to all.

When the girders are not spaced uniformly, or there is only one row of girders, take the width of floor area supported by the girder, for the distance apart. In several cases two sizes are given, both of which have sufficient strength, although one contains less lumber than the other. In most cases the deeper beam has a little excess of strength, but for convenience the shallower beam might be preferred.

TABLE I.

Dimensions of Floor Joists for Different Loads and Spans.

[Note.—The number following the dash denotes the distance apart of joists in inches on centers.]

A.—FOR DWELLINGS.

(Total Weight, 70 lbs. per Square Foot.)

Timber.	Clear Span in Feet.							
	10	12	14	16	18	20	22	24
White Pine.	2 × 8—20	2 × 8—13 2 × 10—24	2 × 10—16	2 × 12—18	2 × 12—13 3 × 12—20	3 × 14—20		
Spruce.	2 × 6—11 2 × 8—24	2 × 8—16	2 × 10—2[illegible]	2 × 10—12 2 × 12—22	2 × 12—15	2 × 12—11 3 × 12—16	3 × 12—23 3 × 14—20	3 × 14—16
Yellow Pine.	2 × 6—16	2 × 8—22	2 × 8—14 2 × 10—24	2 × 10—16	2 × 12—20	2 × 12—16	2 × 12—12 3 × 12—18	3 × 12—14 3 × 14—[illegible]0

B.—FOR HOTELS, SCHOOL-ROOMS, LIGHT OFFICES, ETC.

(Total Weight, 100 lbs. per Square Foot.)

Timber.	Clear Span in Feet.							
	10	12	14	16	18	20	22	24
White Pine.	2 × 8—16	2 × 10—18	2 × 10—11½ 2 × 12—20	2 × 12—13 3 × 12—19	3 × 12—14	3 × 14—16.		
Spruce.	2 × 8—19	[illegible] × 8—1[illegible] 2 × 10—22	2 × 10—13½ 2 × 12—24	2 × 12—16	3 × 12—17	3 × 12—1[illegible] 3 × 14—19	3 × 14—14	
Yellow Pine.	2 × 8—24	2 × 8—15 2 × 10—26	2 × 10—18	2 × 10—12 2 × 12—2[illegible]	2 × 12—15½	3 × 12—16	3 × 12—12 3 × 14—20	3 × 14—15

C.—FOR OFFICE BUILDINGS, ASSEMBLY ROOMS, AND LIGHT STORES.

(Total Weight, 130 lbs. per Square Foot.)

Timber.	Clear Span in Feet.							
	10	12	14	16	18	20	22	24
White Pine.	2 × [illegible]—12 2 × [illegible]—[illegible]2	2 × 10—14	2 × 12—15	3 × 12—15	3 × 14—16			
Spruce.	2 × [illegible]—1[illegible]	2 × 10—17	2 × 12—18½	2 × 12—12 3 × 12—18	3 × 12—13	3 × 14—15		
Yellow Pine.	[illegible]—21	2 × 8—12 2 × 10—23	2 × 10—15	2 × 12—17	2 × 12—12 3 × 12—18	3 × 12—13 3 × 14—21	3 × 14—15	3 × 14—18

D.—FOR STORES AND FACTORIES.*

(Total Weight, 180 lbs. per Square Foot.)

Clear Span in Feet.							
10	12	14	16	18	20	22	24
2×10–16	2×10–11 2×12–1	2×12–11 3×12–17	3×12–18 3×14–18	3×14–14½	3×14–11		
2× 8–12 2×10–17	2×10–12 2×12–18	2×12–13 3×12–20	3×12–16	3×12–12 3×14–16	3×14–18	3×14–11	
2× 8–17	2×10–19	2×10–18 2×12–19	2×12–15	2×12–12 3×12–18	3×12–14 3×14–19	3×14–16	3×14–13½

* Calculated for strength only.

TABLE II. A.

Dimensions of Wooden Girders for Dwellings.

(Total Weight, 70 lbs. per Square Foot.)

SPRUCE.

IN r.	Distance apart on Centers in Feet.							
	10	12	14	16	18	20	22	24
	6×10 8× 8	6×10 8× 8	6×10	8×10	8×10	9×10	8×12 10×10	8×12 10×10
	6×10	6×10	8×10	8×10	8×12 10×10	8×12 10×10	8×12	9×12
	6×10	8×10	6×12 9×10	8×12 10×10	8×12	10×12	10×12	10×12
	8×10	6×12 10×10	8×12	8×12	10×12	10×12	10×14 12×12	10×12 12×14
	6×12 10×10	8×12	10×12	10×12	10×14 11×12	10×14 12×12	10×14	11×14
	8×12	9×12	10×12	10×14 12×12	10×14	10×14	12×14	12×14

YELLOW PINE.

IN r.	Distance apart on Centers in Feet.							
	10	12	14	16	18	20	22	24
	6× 8	6× 8	6×10 8× 8	6×10 8× 8	6×10 8× 8	6×10	8×10	8×10
	6× 8	6×10 8× 8	6×10	6×10	8×10	8×10	8×10	9×10
	6×10	6×10	6×10	8×10	8×10	6×12 10×10	8×12 10×10	8×12
	6×10	8×10	8×10	6×12 10×10	8×12 10×10	8×12	8×12	10×12
	8×10	6×12 10×10	6×12 10×10	8×12	8×12	10×12	10×12	10×12
	6×12 10×10	6×12	8×12	8×12	10×12	11×12	10×14 11×12	10×14
	8×12	8×12	10×12	10×12	10×14 12×12	10×14	10×14	10×14

B.

Dimensions of Wooden Girders for Hotels, School-rooms, Light Offices, etc.

(Total Weight, 100 lbs. per Square Foot.)

SPRUCE.

Span in Feet.	Distance apart on Centers in Feet.							
	10	12	14	16	18	20	22	24
9	6 × 10	8 × 10	8 × 10 6 × 12	8 × 12 10 × 10	8 × 12	10 × 12	10 × 12	10 × 12
10	8 × 10 6 × 12	8 × 10 6 × 12	8 × 12 10 × 10	8 × 12	10 × 12	10 × 12	10 × 14 12 × 12	10 × 14 12 × 12
11	8 × 12 10 × 10	8 × 12	10 × 12	10 × 12	10 × 14 12 × 12	10 × 14 12 × 12	10 × 14	12 × 14
12	8 × 12	10 × 12	10 × 12	10 × 14 12 × 12	10 × 14	12 × 14	12 × 14	14 × 14
13	10 × 12	10 × 12	10 × 14 12 × 12	10 × 14	12 × 14	12 × 14	12 × 16 14 × 14	12 × 16
14	10 × 12	10 × 14 12 × 12	10 × 14	12 × 14	14 × 14	12 × 16	12 × 16	14 × 16
15	10 × 14 12 × 12	10 × 14	12 × 14	12 × 16 14 × 14	12 × 16	14 × 16	14 × 16	16 × 16

YELLOW PINE.

Span in Feet.	Distance apart on Centers in Feet.							
	10	12	14	16	18	20	22	24
9	6 × 10 8 × 8	6 × 10 8 × 8	6 × 10	8 × 10	8 × 10	8 × 12 10 × 12	8 × 12	8 × 12
10	6 × 10	6 × 10	8 × 10	8 × 10	10 × 10	10 × 10	8 × 12	10 × 12
11	6 × 10	8 × 10	6 × 12 10 × 10	8 × 12 10 × 10	8 × 12	10 × 12	10 × 12	10 × 12
12	8 × 10	10 × 10	8 × 12	8 × 12	10 × 12	10 × 12	10 × 14 12 × 12	10 × 14 12 × 12
13	8 × 12 10 × 10	8 × 12	10 × 12	10 × 12	10 × 14 12 × 12	10 × 14	10 × 14	12 × 14
14	8 × 12	10 × 12	11 × 12	10 × 14 12 × 12	10 × 14	10 × 14	12 × 14	12 × 14
15	10 × 12	10 × 14 12 × 12	10 × 14 12 × 12	10 × 14	12 × 14	12 × 14	12 × 16 14 × 14	12 × 16 14 × 14
16	10 × 14 12 × 12	10 × 14	10 × 14	12 × 14	13 × 14	10 × 16 14 × 14	12 × 16	12 × 16

C.

Dimensions of Wooden Girders for Office Buildings, Assembly Rooms, and Light Stores.

(Total Weight, 130 lbs. per Square Foot.)

SPRUCE.

Span in Feet.	Distance apart on Centers in Feet.							
	10	12	14	16	18	20	22	24
9	8 × 10	8 × 12 10 × 10	8 × 12	10 × 12	10 × 12	10 × 14 12 × 12	10 × 14 12 × 12	10 × 14
10	8 × 12 10 × 10	8 × 12	10 × 12	10 × 14 12 × 12	10 × 14	10 × 14	12 × 14	12 × 14
11	8 × 12	10 × 12	10 × 14 12 × 12	10 × 14	10 × 14	12 × 14	13 × 14	14 × 14
12	10 × 12	10 × 14 12 × 12	10 × 14	12 × 14	12 × 14	12 × 16 14 × 14	12 × 16	14 × 16
13	10 × 14 12 × 12	10 × 14	12 × 14	12 × 16 14 × 14	12 × 16	13 × 16	14 × 16	
14	10 × 14	12 × 14	12 × 16 14 × 14	12 × 16	14 × 16	15 × 16		

YELLOW PINE.

Span in Feet.	Distance apart on Centers in Feet.							
	10	12	14	16	18	20	22	24
9	6 × 10	8 × 10	8 × 10	6 × 12 10 × 10	8 × 12 10 × 10	8 × 12	8 × 12	10 × 12
10	8 × 10	8 × 10	8 × 12 10 × 10	8 × 12	8 × 12	10 × 12	10 × 12	10 × 14 12 × 12
11	8 × 10	8 × 12 10 × 10	8 × 12	10 × 12	10 × 12	10 × 14 12 × 12	10 × 14 12 × 12	10 × 14
12	8 × 12	8 × 12	10 × 12	11 × 12	10 × 14 12 × 12	10 × 14	12 × 14	12 × 14
13	8 × 12	10 × 12	10 × 14 12 × 12	10 × 14	10 × 14	12 × 14	14 × 14	14 × 14
14	10 × 12	10 × 14 12 × 12	10 × 14	12 × 14	12 × 14	14 × 14	12 × 16	12 × 16
15	10 × 14 12 × 12	10 × 14	12 × 14	12 × 14	12 × 16 14 × 14	12 × 16	14 × 16	14 × 16

D.

Dimensions of Wooden Girders for Stores and Factories.

(Total Weight, 180 lbs. per Square Foot.)

SPRUCE.

Span in Feet.	Distance apart on Centers in Feet.							
	10	12	14	16	18	20	22	24
9	8 × 12	10 × 12	10 × 12	10 × 14 / 12 × 12	10 × 14	12 × 14	12 × 14	10 × 16 / 14 × 14
10	10 × 12	10 × 14 / 12 × 12	10 × 14	12 × 14	12 × 14	10 × 16 / 14 × 14	12 × 16	14 × 16
11	10 × 14 / 12 × 12	10 × 14	12 × 14	10 × 16 / 14 × 14	12 × 16	13 × 16	14 × 16	
12	10 × 14	12 × 14	14 × 14	12 × 16	14 × 16			
13	12 × 14	14 × 14	12 × 16	14 × 16				

YELLOW PINE.

Span in Feet.	Distance apart on Centers in Feet.							
	10	12	14	16	18	20	22	24
9	8 × 12	8 × 12 / 10 × 10	8 × 12	8 × 12	10 × 12	10 × 12	10 × 14	12 × 14
10	8 × 12 / 10 × 10	8 × 12	10 × 12	10 × 12	10 × 14 / 12 × 12	10 × 14	11 × 14	12 × 14
11	8 × 12	10 × 12	10 × 14 / 12 × 12	10 × 14	10 × 14	12 × 14	13 × 14	11 × 14
12	10 × 12	10 × 14 / 12 × 12	10 × 14	12 × 14	12 × 14	14 × 14	12 × 16	13 × 16
13	10 × 14 / 12 × 12	[illegible] × 14	12 × 14	13 × 14	12 × 16 / 14 × 14	12 × 16	13 × 16	15 × 16
14	[illegible] × 14	12 × 14	14 × 14	12 × 16	13 × 16	14 × 16	15 × 16	

JOIST HANGERS.

FIG. 4.—DUPLEX JOIST HANGER.

FIG. 5.—GOETZ JOIST HANGER.

Figs. 4 and 5 show two styles of joist hangers that have been put on the market within a few years. Both these anchors are warranted to be stronger than the timber they support. They are made in numerous sizes, and are inserted in holes bored in the sides of the girder, or trimmer. While these hangers themselves, however, have ample strength, they must weaken to some extent the timber into which the holes are bored, which is not the case with the stirrup iron.

Fig. 6 shows a similar hanger made to support the wall end of floor joist. The writer believes this to be much superior to the method of building the joist into the wall, as it absolutely prevents dry rot, and permits the joist to fall in case of fire, without throwing the wall. It also gives the weight a good bearing on the wall.

FIG. 6.—DUPLEX BRICK WALL HANGER.

CHAPTER XXIII.

FIRE-PROOF FLOORS.

THE term "fire-proof floor" is here understood to mean a floor constructed of fire-proof material, supported on or between iron or steel beams or girders, or fire-proof walls, and entirely protecting the ironwork from the action of fire. The various materials at present used in the construction of absolutely fire-proof floors are brick, hollow porous tile, hollow dense tile, thin plates of dense tile

Fig. 1.

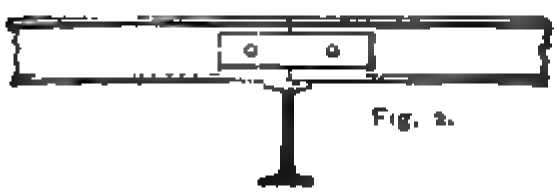


Fig. 2.

— all products of clay; and concrete of Portland cement and either cinders, broken tile, stone, or brick; and also compositions made with plaster of Paris as a cementing material. The first three materials are generally used in the form of arches set between the beams. The thin plates of dense tile are used for forming vaults between girders. Concrete is used either in the form of an arch or in flat slabs forming floor and ceiling with hollow interior; in the slabs iron bars, expanded metal, or wire ties are imbedded. Iron or steel beams are generally laid in floors as shown in Fig. 1, the joists either resting on top of the girders, as in Fig. 2, or bolted to the sides of the girders.

Fig. 3 shows the detail of connection when the under sides are made flush; Fig. 4, the joint to bring the upper sides flush; and Fig. 5 shows the form usually adopted when the beams are of the same size, or the centre lines are brought together. Arrangements of this kind are also used to connect the trimmer-beams of hatchways, jambs, and stairways.[1]

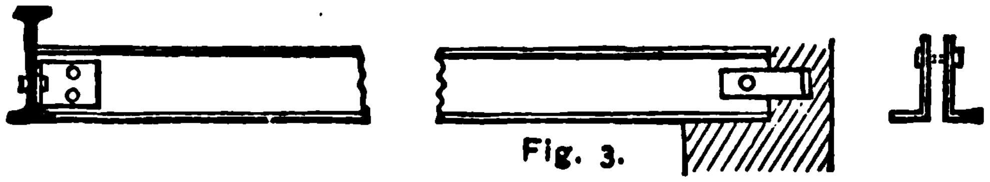


Fig. 3.

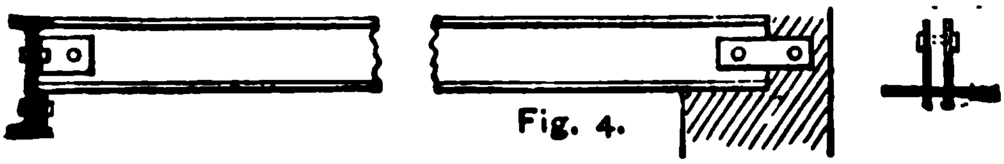


Fig. 4.

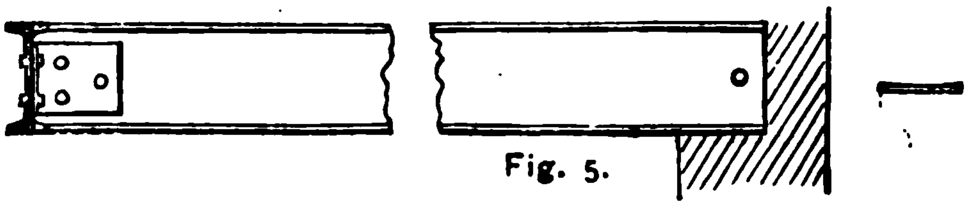


Fig. 5.

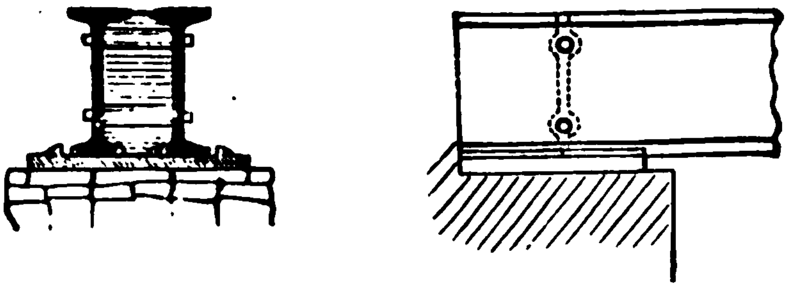


Fig. 6.

The wall ends of the joists and girders should be provided with *shoes or bearing* plates of iron or stone, as the brickwork is apt to crush under the ends of beams, unless the load is distributed by this means over a sufficient surface. *Anchor-straps* should be bolted *to the end of each girder* and *to the wall end* of every alternate joist, binding the walls firmly from falling outwards in the event of fire or other accident.

Several simple modes of anchorage are shown in Figs. 3, 4, and 5.

When one beam does not give sufficient strength for a girder, it is customary to bolt together two or more with cast separators between them, as shown in Fig. 6.

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[1] The details of the connections and framing of iron beams are more clearly shown on pp. 365, 366.

Brick Arches.

The common way of making a fire-proof floor of brick is to fill the space between the joists with brick arches, resting on the lower flanges against terra-cotta or brick skewbacks. When this method is pursued, care should be taken that the bricks of which the arches are composed are of good shape, and very hard. They should be laid in contact with each other, without mortar; and all the joints should be filled with the best cement grout, and be keyed with slate

Fig. 7.

The arches need not be over four inches thick for spans between six and eight feet, except for about a foot at each springing, where they should be eight inches thick for spans between six and eight feet, care being taken to form the skewbacks quite solid, and square to the line of pressure. The rise of the arch should be about one-eighth of the span, or an inch and a half to the foot;[1] and the [illegible] spans between four and six feet.

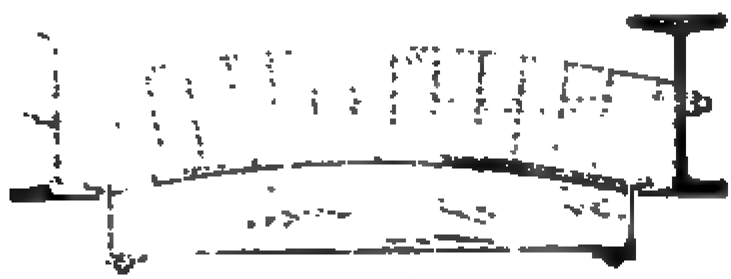


Fig. 8.

[illegible] concrete, in which [illegible] [illegible] [illegible] to one inch [illegible] feet apart, and [illegible] of the building to the [illegible] with stout washers; an

[illegible]

angle-bar or channel serving as a wall plate for distributing the strain produced by the thrust of the first arch (Fig. 7).

The weight of a brick arch with cement filling is about seventy pounds per superficial foot of floor. Experience has shown that such a floor cannot be considered as fire-proof unless the lower flanges of the beam are protected by porous terra-cotta, fire-clay tile, or wire lathing, kept an inch away from the beam.

Brick floor arches are largely going out of use, owing to the fact that a fire-proof floor may be more cheaply constructed of other material.

Hollow Porous Terra-cotta and Hollow Dense Terra-cotta Floors.—For convenience, these materials will be referred to as Porous Tiling and Dense Tiling. A description of the materials, their nature and manufacture, will be found in Chapter XXV. They consist principally of clay, which is manufactured into hollow blocks, generally with angles on side or ends, according to whether the arches of the floors are to be of end-method design or side-method design. In some instances, to a limited extent, rectangular blocks have been successfully used, but this shape is not approved. The general practice in flat construction is to make bevel joints—radius joints are seldom used; the best workmanship and best results are found to be obtained with a bevel joint of about one inch to the foot. There are two general schemes of flat construction: one in which the tile blocks abut end to end continuously between the beams, and one in which they lie side by side, with broken joints, between the beams. In the end systems, it is not the practice to have the blocks in one row break joints with those in another, as it entails extra expense in setting. When this is done, however, the substantialness of the floors is increased.

In some forms of flat construction a side-method skewback (or abutment) is used, with end-to-end interiors and keys, or end-to-end interiors and side-method keys. Experience has shown that in the side method of flat construction the skewback, or abutment, was the weakest—in case of failure, sometimes collapsing, but generally shearing off at the beam flange; consequently, the side-method skewback is not approved in the end-method construction unless provided with partitions running at right angles to the beam. Keys should be end to end, or solid. The latter, when made very small, are preferable.

A practice which has become somewhat general, especially in the East, is for the owner or general contractor to buy tiles, and the mason contractor on the job to build them in place in the building.

beams, and like centrepieces above, crossing the beams. The planks on which tiles are laid should be two-inch, dressed on one side to uniform thickness, and should lie on lower centres, at right angles to beams and placed close together The soffit tile should be a separate key-shaped piece, of equal width of beam, and laid directly under the beam on the planking after which the centring is tightened by screwing down the nuts on the T-bolts, until the soffit tile are hard against the beams and the planking has a crown not exceeding one-fourth of an inch in spans of six feet. This system gives what is very essential—a firm and steady centre on which to construct the flat tile work. The tiles should be "shoved" in place with close joints, and keys should fit close The centres should remain from twelve to thirty-six hours, according to conditions of weather, depth of tiling, and mortar used. When centres

Fig. 13.

are "struck," the ceiling should be straight, even, free from open joints, crevices, and cracks, ready to receive the plastering.

Figs. 9 to 12 show types of flat constructions in use. Different manufacturers have various modifications of these. Fig 9 is the most general design for dense tiling, although porous tiling, very similar in design, may be had from some manufacturers. The end-method design is preferable however, for porous tiling Fig. 10 is a light-weight dense-tile design, not so generally used as formerly Figs 11 and 11*b* show the simplest end-method design for porous tiling, which has become known as the "Lee end-method arch." It was first brought into general use by Mr. Thomas A Lee, now of New York City It was used by him in the tests conducted at Denver in December 1890, by Messrs. Andrews, Jaques & Rantoul, architects In those tests the design showed superiority over the other designs. It has the advantage of simplicity and economy, both in manufacture and construction. The manufacturer can

reduced and the stability of construction increased. The reduction in price of all tiling makes the cost rather in favor of increasing the thickness of tiling and reducing the thickness of concrete.

Among the advantages possessed by hollow tiles in their application to fire-proof floors, between steel or iron beams, are these :

They are absolutely incombustible, because made of clay and having withstood a white heat in the course of manufacture.

They are sound-proof, from fact of being hollow.

They are superior to any concrete material used for the same purpose, owing to their being free from shrinkage, thereby avoiding the unsightly cracks often seen in ceilings laid with concrete blocks.

They are proof against rats and vermin.

Floors made of them are forty per cent. lighter than by the old system of segmental solid brick arches levelled with concrete.

They offer a flat surface on the bottom and top after being laid,

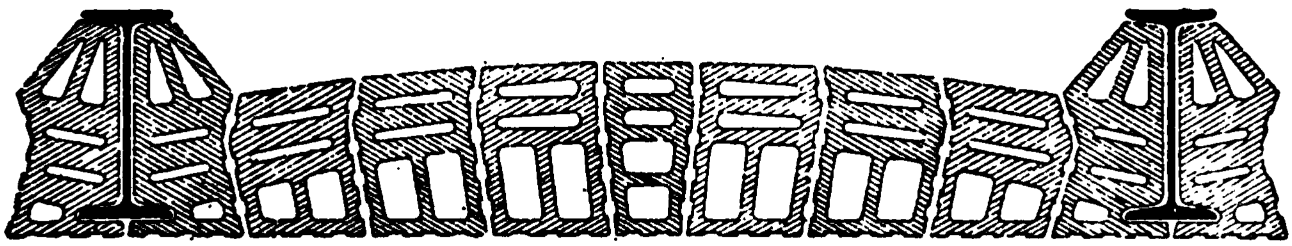


FIG. 16.—"AUSTRIA" ARCH, PATENTED BY FR. VON EMPERGER.

thereby giving a flat ceiling ready for plastering, and a flat foundation for the floor strips.

The flat arches should in all cases be capable of sustaining, without injurious deflection, after being set in place, an equally distributed load of 500 pounds upon each superficial foot of surface.

In laying the tile, a mortar composed of lime mixed up with coarse screened sand, in proportions of four to one, and richly tempered with hydraulic cement, should be used. This makes a strong mortar, and works well with the tile. In no case should a joint exceeding one-half inch in thickness be permitted

The laying of flat construction in winter weather without roof protection should not be practised in climates where frequent severe rain and snow storms are followed by hard freezing and thawing, as the mortar joints are liable to be weakened or ruptured, resulting in more or less deflection of the arches.

The upper surface of these arches is generally covered with concrete of a sufficient depth to allow for bedding in it the wooden strips to which the floor boards are nailed. The concrete can be made of light and cheap materials, such as lime or native cement and clean rolling-mill cinders, coke screenings, broken fire-proof

tiling, etc. The floor strips should be of sound and seasoned wood, 2 inches thick by 2 inches wide on top, bevelled on each side, to 4 inches wide on bottom, placed about 16 inches on centres The concrete should be firmly bedded beneath and against each side. Instead of concrete filling, etc., a filling is sometimes made by laying hollow partition blocks on top of the arches. These form excellent foundations for marble or other finished tile flooring.

The practice of putting in comparatively thin flat arch construction to form ceilings, then heavy wood strips from beam to beam to carry the weight of the floor, leaving a hollow space between top of arches and under side of wood flooring, is not approved. The amount of wood contained in such a floor is sufficient to produce a very damaging heat. The hollow space enables the wood to burn readily, and makes a fire very difficult to fight. Such construction,

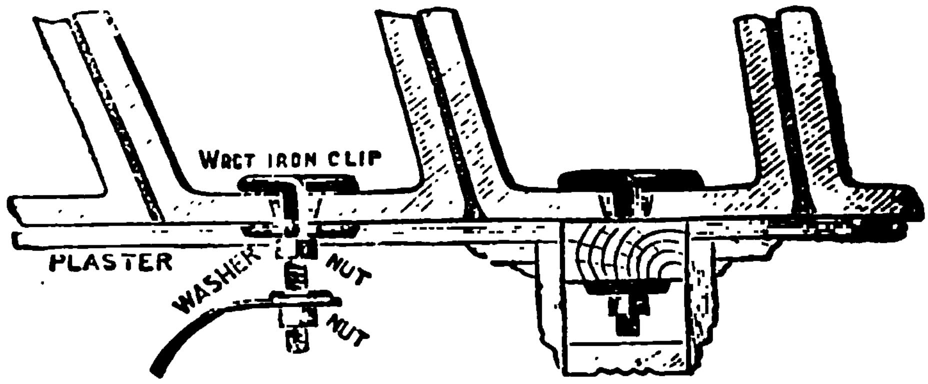


FIG. 17.

therefore, is dangerous, and should not be considered as first-class fire proof work.

The variation in width of spans between beams is provided for by supplying tiles of different sizes, both for interiors and keys, whereby a variety of combinations can be secured.

When desired to attach iron or wood work to the soffits of the hollow tile floor arches, slot holes are punched in the tiles, and T-headed bolts are inserted and secured as shown in Fig. 17.

When porous terra cotta tile are used, cleats may be nailed or screwed directly to the tile.

In setting iron work, too great care cannot be exercised that all beams be placed parallel, especially where one or both ends of beams rest on brickwork. Beams placed out of parallel make it very expensive to set the fire proofing, often requiring cutting of tiles, which is damaging and injurious, and should not be done.

Wide span, segmental hollow tile arches (see Figs. 18 and 19) are sometimes used in warehouses, factories, and for roofs, in thicknesses of six and eight inches. Usually the tiles are 6 x 6 inches,

or 6 × 8 inches, and 12 to 16 or 18 inches long. Spans may be any width up to 20 feet, rise about one inch to foot of whole span. In some instances the joints are pointed after the centres have been removed, and the whole under side painted. This form of hollow-tile work in wide spans from girder to girder is cheaper and lighter than flat construction with floor beams.

4" to 6" Segmental Arch Weight of tile 26 to 36 Lbs. Span 6'0" to 20'0" according to size of beam.

FIG. 18.

Weights and Safe Spans for Dense-tile Arches.—The following table gives the weight and span of flat hollow dense-tile arches made by the Raritan Hollow and Porous Brick Company. This is about an average for spans given by different manufacturers. The Pioneer Fire-proof Construction Company, and some others, make a lighter grade of tile than this, but their heavy tiles correspond very closely with the table below. Dense tiles may also be had from Lorillard Brick Works Company and Henry Maurer's Son, New York; the Empire Fire-proofing Company, Pittsburg; Parker & Russell Company, St. Louis; and others.

WEIGHTS AND SPANS OF FLAT HOLLOW DENSE-TILE ARCHES.

Depth of Arch.	Span, between Beams.	Weight per sq. ft.
6 in.	3 ft. 6 in. to 4 ft.	29 lbs.
7 in.	4 ft. to 4 ft. 6 in.	32 lbs.
8 in.	4 ft. 6 in. to 5 ft. 6 in.	35 lbs.
9 in.	5 ft. to 5 ft. 9 in.	37 lbs.
10 in.	5 ft. 9 in. to 6 ft. 6 in.	41 lbs.
12 in.	6 ft. 6 in. to 7 ft. 6 in.	48 lbs.

The following table gives the weight and span of flat hollow porous-tile arches of the Lee end-method design, which may be

olted together with $\frac{3}{4}$-inch tie rods, secured to the web of the eams near the bottom flanges. and drawn tightly to place by nut nd thread. These tie rods should be set from five to seven feet part.

The cost of hollow-tile arches of either kind, set in place ready or plastering, in lots of 20,000 square feet, ranges from 14 cents to 5 cents per square foot, according to size and weights of the tile. n Chicago the average price may be taken at 20 cents.

Specifications for Transverse System of End-Pressure Floor Arch.

The following form of specification may be of assistance to rchitects in preparing their specifications for tile floors :

Contractors submitting proposals for fire-proof floor arches shall, hen required, prepare detail drawings showing the system and pplication of floor arch proposed to be used. The general requirements of such design shall be as follows :

1st. Arches to be level top and bottom, filling space between the eams from a point not less than seven-eighths of an inch below e soffit of beam up to within one inch of the top of the beam.

2d. The abutment tile adjoining or resting upon the floor beams hall have its hollows run parallel with the beams, but the vousoirs shall be laid transversely, with hollows running at right ngles to the floor beams, so that the tile blocks forming the arch ay receive the pressure resulting from imposed load on their end ection and distribute it lengthwise of their respective web members.

3d. Soffits of all beams shall be covered with tile slabs keyed ecurely in place, flushing with under surface of arch.

TESTS.

Each arch shall be subjected to a test of a moving load consisting f a roller weighing 1,000 pounds to each lineal foot, and applied orty-eight hours after the centres have been struck and before the oncrete has been filled in. This roller to be rolled over the top of he tile wherever the supervising architect or his superintendent hall direct.

In addition to such rolling test, the arches, after being set in lace seventy-two hours, shall be subjected to a dropping test made n the following manner : Before the concrete is applied on the rches, a bed of sand two inches thick shall be spread loosely over e top of the arches, and a wooden block or timber, weighing 200 ounds, shall be dropped thereon from a height of ten feet. If the

arches withstand this impact for three continuous blows without breaking through, the test shall be considered satisfactory, and the floor arches be accepted. Should the floor arches break through under the blows, it shall be deemed conclusive that the method of floor arch employed is faulty, and the contractor will be required to remove same from the building and provide arches suitable to withstand the tests required.

Strength of Flat Hollow Dense and Porous Terra-Cotta Arches.—Either of these materials, when properly made and erected, should have a strength of at least 500 lbs. per square foot. One of the most complete and practical tests of floor arches recorded was made in Denver, Col., under the direction of Messrs. Andrews, Jaques & Rantoul, architects, for the Denver Equitable Building Company, December 20-23, 1890, of which a full report was published in the *American Architect and Building News*, March 28, 1891. Eight arches built of hollow burned fire-clay tile, and four of porous terra-cotta, were subjected to four kinds of tests, under as nearly the same conditions as possible. The arches were carried on 10-inch steel I-beams, set 5 feet apart on centres, and were built of 10-inch tile. The terra-cotta tile were manufactured by Mr. Thomas A. Lee, and were of the end-construction type shown in Figs. 11 and 11*b*, and it is doubtless owing to this fact that these arches developed the strength shown by the tests.

The tests were as follows:

1st. By still load, increased until the arches broke down.

2d. By shocks, repeated until the arches were destroyed.

3d. Tests by fire and water, alternating until the arches were destroyed.

4th. By continuous fire of high heat, until the arches were destroyed.

Under the first test, one of the fire-clay tile arches broke at 5,42[illegible] lbs., or 271 lbs. per square foot, and the other at 8,574 lbs., or 128 lbs. per square foot; both of these arches had but one horizontal web, which was at the centre of the tile. Both of these arches gave way suddenly, the whole arch falling down, the failure in both cases taking place in the skewbacks, the remainder of the arch being uninjured. The porous terra-cotta arch, which had two horizontal webs, sustained a load of 15,145 lbs., [illegible] lbs. per square foot, for [illegible] hours without breaking, when the load was discontinued.

The second series of tests was made by dropping a piece of timber 12 inches square and 4 feet long, weighing 134 lbs., from a height of six feet, upon the middle of the arch. Both of the hol-

low fire-clay tile arches broke at the first blow of the ram, the arches dropping from between the beams, the tile breaking "like a sheet of glass, indicating extreme brittleness in the material."

The porous terra-cotta arch withstood four blows from a height of six feet, and seven blows from a height of eight feet, the arch dropping at the last blow. Pieces of one or more of the tile, however, dropped out at nearly every blow. Under the fire and water test, one of the fire-clay arches was destroyed by three applications of the water; the other withstood fourteen applications of the water, alternating with extreme heat.

The porous terra-cotta arch withstood eleven applications of water, alternating with extreme heat, uninjured. The temperature of the tile at the time the water was applied varied from 1,300° to 1,600° F. Under the continuous fire test, both fire-clay arches were destroyed after being subjected to a most intense heat for twenty-four hours. The porous terra-cotta arch, after having a continuous fire under it for twenty-four hours, was practically uninjured, as it afterward supported a weight of bricks of 12,500 lbs. on a space 3 feet wide, in the middle of the arch.

These tests were conducted with perfect fairness, and unquestionably show the superiority of the porous terra-cotta arches. The porous terra-cotta tile, new and dry, weighed 34 lbs. to the square foot; the fire-clay tile which stood the tests the best weighed 40¾ lbs. per square foot, and the other 32 lbs. per square foot.

Other Tests.—During the construction of the Board of Trade building, in Chicago, in 1884, a 6-inch tile arch of 3 feet 8 inches span, made by the Wight Fire-proofing Company, of Chicago, was loaded up to 706 lbs. per square foot without injuring the arch. The arch was also severely tested by dropping heavy dry-goods cases upon it from a height of 4 feet, without injury.

When the large (16-feet) span arches were laid in the Commerce building, on Pacific Avenue, in Chicago, each arch was tested by rolling an iron pulley, 6 feet in diameter and 14 inches wide, weighing 2,180 lbs., over each square foot, before the concrete had been filled in the haunches. This is a convenient method of testing the strength of a floor after it is laid, and its use is to be highly recommended.

Strength of Brick Arches.—Brick arches, properly built between iron beams, as described on page 440, are practically indestructible, from any usage or load that could occur in a building.

When the Western Union Telegraph building, in New York City, was being erected, Mr. F. C. Merry, the architect, made a series of tests on several forms of floor arches, supported by iron

beams placed about five feet apart, by dropping a piece of granite, 15 inches square and 4 feet long, with rounded edges, from a height of three feet, on top of the arches: and, while all of the other arches were destroyed, the brick arch withstood the shock several times uninjured, and only after repeated poundings in the same place one brick at a time was knocked out until the arch was finally broken down.

That brick floor arches will endure great distortion was shown by the loading of an arched floor at the Watertown Arsenal, Mass. A floor 29 feet square, was made of five 15-inch I-beams, 200 lbs. per yard, carrying brick arches. The beams were 7 feet 4.8 inches apart on centres, and rested on brick walls 28 feet 6 inches apart. The rise of the brick arches was 8.5 inches. "Common, rather soft-burned brick were used, laid on edge with lime mortar. The arches were backed with concrete, and planked over. The maximum load carried by this floor (when the beams, and not the arches, failed) was 563 lbs. per square foot. This load caused a gradual and continuous yielding of the beams, which was allowed to continue till the floor was deflected a distance of 13.07 inches, measured at the centre of the middle beams." "The brickwork endured this great deflection, and apparently would have stood much more without failure," had it been possible to carry the test further.*

Fire-Proof Floors with Tension Members (1895).—Within a few years several styles of fire-proof floor construction have been introduced, of which there are two general classes; the first class consists of tension member floors, which in themselves furnish the necessary strength for sustaining the floor from wall to wall, or wall to girder, without the use of floor beams; and the other class consists of I-beams five or six feet apart for sustaining the floor, with rods or bars suspended or resting upon the beams, supporting wire cloth, netting, or expanded metal, which carries the concrete or plaster filling. Prominent among the first devices mentioned are the Hyatt ribbed metal ties and Portland cement concrete floors built by P. H. Jackson, San Francisco; the concrete and twisted bar floors built by the Ransome & Smith Company, of Chicago; and the Lee hollow tile and cable rod floors, built by the Lee Fire-proof Construction Company, of New York.

Prominent among the I-beam and concrete filling devices are the systems of the Metropolitan Fire-Proofing Company, of Trenton, N. J.; the expanded metal construction companies of St. Louis

* J. E. Howard, in *American Architect and Building News*, March 10, 1888.

and New York; and the New Jersey Wire Cloth Company, of Trenton, N. J.

Hyatt and Jackson Concrete Floors.—Concrete composed of broken stone, fragments of brick, pottery, and gravel, held together by being mixed with lime, cement, asphaltum, or other binding substances, has been used in construction to resist compressive stress for many ages.

With the introduction of Portland cement, concrete construction has taken a more important position among the various methods of building, so that now entire buildings are constructed of concrete, such as the Hotel Ponce de Leon at St. Augustine, Florida; and in California, especially, concrete is largely used in the construction of floors, sidewalk arches, etc.

The concrete is not used between iron beams, as are the brick and tile arches, but the concrete itself is made self-supporting from wall to wall by means of embedding iron in the bottom of the concrete. Portland cement concrete has a great resistance to compression, but possesses little tensile strength.

In 1876 Mr. Thaddeus Hyatt, the inventor, while considering the matter of fire-proof floor construction, conceived the idea of forming concrete beams by embedding iron in the bottom of the concrete to afford the necessary tensile strength which the concrete lacked. Mr. Hyatt made many experimental beams, with the iron introduced in a great variety of ways, as straight ties, with and without anchors and washers; truss rods in various forms; flat pieces of iron set vertically and laid flat, anchored at intervals along the entire length. These experimental beams were tested and broken by David Kirkaldy, of London, and the results published by Mr. Hyatt for private distribution, in the year 1877.

By these tests Mr. Hyatt proved conclusively that iron could be perfectly united with concrete, and that it could be depended upon under all conditions for its full tensile strength.

The method Mr. Hyatt adopted as the best for securing perfect unison of the iron and concrete was to use the iron as thin vertical blades placed near the bottom of the concrete beam or slab, extending its entire length, and bearing on the supports at both ends;

Fig. 14.

these vertical blades to be anchored at intervals of a few inches by round wires threaded through holes punched opposite each other in the vertical blades, thus forming a skeleton or gridiron, as shown in Fig. 14. For a perfect combination of these substances, it is essential that the one should be united with the other in such a manner that the iron cannot stretch or draw without the concrete extending with it.

The only person in this country to make practical application of the method devised by Mr. Hyatt, so far as the author is aware, is Mr. P. H. Jackson, of San Francisco, Cal., who has used it quite extensively in that city for covering sidewalk vaults, and for the support of store lintels; also, for self-supporting floors. Mr. Jackson published a pamphlet in 1890, entitled *Improvement in Building Construction*, which gives a great amount of information on this subject, and on concrete in general construction.

To show the strength of this method of construction, Mr. Jackson, in August, 1885, prepared a beam, 7 × 14 inches in section and 10 feet 6 inches long; near the bottom were seven vertical blades of iron extending the entire length; three of these were $\frac{1}{4} \times 1$ inch, and four were $\frac{1}{8} \times 1$ inch, with $\frac{1}{4}$-inch wires threaded through every 3 inches. Near the top were bedded two cast-iron rope moulding bars to assist the compressive strength of the concrete, which, however, was shown to be unnecessary. The concrete at the top and bottom was one part cement to one of sand; centre portion, one of cement to two of sand. The beam was supported by 9-inch bearings at both ends, thus leaving it 9 feet in the clear between supports. The beam was loaded with pig-iron piled across it, and broke under a load of 53,654 lbs., by separating all the longitudinal blades on the line of one of the cross-wires near the centre. Just before breaking, the deflection was measured, and found to be $1\frac{3}{8}$ inch. The breaking load of this beam was about one-half that which would have broken a hard-pine beam of same dimensions and average quality.

The Ransome and Smith Floor.

While Mr. Jackson was experimenting with the Hyatt ties, Mr. E. L. Ransome, a very successful worker of concrete in San Francisco, conceived the idea of using square bars of iron and steel, twisted their entire length, in place of the flat bars and wires used by Mr. Jackson, as shown in Fig. 15. It was found that these bars were held in the concrete equally as well, if not better than the other, and that they were much less expensive. None of the iron

in the ties is wasted, and it has been demonstrated by careful experiments that the process of twisting the bars to the extent desired strengthens the rods instead of weakening them.

Fig. 15.

Mr. Ransome patented his improvement in 1884, and since that time it has been used quite extensively in San Francisco.

The bars, preferably made from the best quality of rectangular iron, are twisted at an expense not exceeding from twenty-five to fifty cents per ton, which constitutes an insignificant item of cost. The sizes so far used range from $\frac{1}{4}$ inch to 2 inches square.

Concrete floors, as made by Mr. Ransome, are made in two forms —flat, and recessed or panelled.

It can be and has been used for spans up to 34 feet. A section of a flat floor, in the California Academy of Science, 15 × 22 feet, was tested in 1890 with a uniform load of 415 lbs. per square foot, and the load left on for one month. The deflection at the centre of the 22-feet space was only [illegible] inch. It was estimated by the architects that the saving in this construction over the ordinary use of steel beams and hollow-tile arches of the same strength, and with similar cement-finished floors on top, amounted to 50 cents per square foot of floor. As a fire-proof construction, the concrete and iron construction above described is undoubtedly equal to any other construction in use.

Composition of the Concrete.—Regarding the concrete used for these floors, the proportions are given for a cement of good average quality, that will develop a tensile strength of 350 lbs. per square inch in fourteen days. If a weaker cement is used, the quantity should be proportionately increased.

The aggregates should be of any of the following substances, which are named about in the order of merit, the first being the best: Hard limestone rock, hard clinker brick, hard broken pottery, granite or basalt, hard clinkers, broken flint or other hard rock, gravel.

Care should be taken to use neither dirty nor soft clayey rock. The aggregates should be broken so as to pass through a two-inch

ring, and the fine dust removed by washing or screening (washing preferred). In mixing add sufficient water to bring the mass into a soft, pasty condition, and tamp it thoroughly into place.

On the bottom of the mould place about one inch and a half of concrete made of one part cement to two parts of aggregates varying from $\frac{1}{16}$ to $\frac{1}{4}$ inch in diameter. Lay the lower iron bars on this mixture and tamp them down into it; then fill up with a concrete composed of one part cement and six parts aggregates, making the final layer of double strength.

The Lee Hollow Tile and Cable Rod Floor.

Fig. 22 is a sketch typical of the Lee Hollow Tile and Cable Rod Floor, with a finished cement top. The floors are usually designed on a basis of $\frac{3}{4}$ inch in depth for each foot of span. The spans extend from wall to wall or from girder to girder, no I-beams being used.

Hollow porous terra-cotta tiles having square ends and a rod

Fig. 22

groove along one side near the base, are used. These tiles are similar to the Lee end arch tiles. Temporary forms carried on horses are provided, and the tiles are laid with Portland cement mortar in rows, end to end, from wall to girder, or from girder to girder. Into the groove of each row of tiles soft cement is placed, and one or more rods, according to strength requirements, are buried in the soft cement. The process is repeated until the whole floor is formed. The rods stop at ends of the tiles at wall lines. Anchor bolts are provided, as indicated, for tying the floor to the supports, and for tying the building together.

By this method of construction, which may be applied to filling between I-beams, all thrust is taken up by the cable rod, and each tile is bound in its place. Cracks, deflections, and other defects often attending flat arches are avoided. The floors are firm, rigid, and stiff. The floors are based upon the transverse strength of

beams. Computations, verified by actual tests, are made, and the use of needless material and weight is thereby avoided.

The cable rods used in the Lee system are made of round drawn steel rods of about thirty one-hundredths of an inch in diameter, laid spirally together, usually in two strands, as that form affords large gripping surface for the cement. Mr. Lee's patents cover a variety of forms, some containing several strands, with different shaped buttons, washers, etc., for affording great cement engaging surface. The rods being of drawn steel, they have high tensile strength, and are specially free from flaws or defects; hence are found to make excellent tension members. The rods are spaced 8, 10, or 12 inches apart, according to width of tile used. The widths and shapes of tiles are varied to suit different spans and loads.

Fig. 23 shows one design of roof for ten-foot spans. It is a

Fig. 23

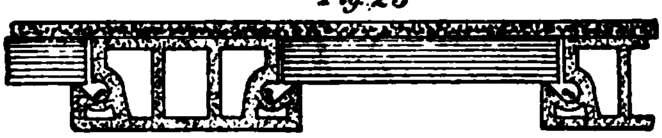


Fig. 24

special adaptation of the system, to cases requiring large protection to the metal from heat, as in dust chambers of smelters.

Fig. 24 shows light design with finished wood top, suitable for dwellings, the wood top being more expensive than cement top. With a cement top the completed structure is but little more expensive than a wood joist structure for the same purpose. The floors are absolutely incombustible, sound-proof, and vermin-proof.

Strength and weight tables are furnished by the builders, giving various depths of floor structures for different spans and loads.

The Metropolitan Company's Floors.

Under this system, which has heretofore been known as the "Manhattan" system, and is protected by letters patent, fire-proof floors are made as follows:

Cables, each composed of two galvanized wires, twisted, are placed at given distances apart over the tops of the beams and transversely with them, as shown in Fig. 25. These cables pass under bars in the centre of the spans, and are thus given a uniform deflection between each pair of beams. The distance between the cables is varied with the loads to be provided for. Forms or centres are then placed under them, and a composition, made principally of plaster of Paris and wood chips, is poured on. This composition solidifies in a few minutes, after which the forms or

centres are removed. The resulting floor is sufficiently strong to be used at once under the loads for which it has been calculated, and as its surface is uniform and level with the tops of the beams, a working floor is thus furnished. This is of great advantage in facilitating the general construction of buildings.

FIG. 25.

Fig. 26 shows the arrangement employed in cases where a flat ceiling is not required. In this arrangement the under side of the floor-plate furnishes a ceiling surface ready for plastering. The lower portions of the beams, projecting as they do below the floor-

FIG. 26.

plates, are protected with a covering of the composition cast in place. Imbedded in this is wire netting passing under each beam and attached to the cables which carry the floor-plates.

Fig. 27 shows the arrangement employed where a flat ceiling

is desired. In this case the floor-plate is the same as in Fig. 26. The ceiling-plate is formed as follows: Bars are placed upon the lower flanges of the beams, and on these wire netting is laid. Centres are placed one inch below the beams, and the composition is poured thereon. The centres are then removed, and the ceiling thus made is ready for plastering. Whether a ceiling like that shown in Fig. 26, or a flat ceiling as shown in Fig. 27, is used, the webs of all beams are covered with about three inches in thickness of the Metropolitan composition, which thoroughly protects the beams from the effects of heat. It is claimed that this material is so remarkable a non-conductor of heat that a moderate thickness of it prevents the passage of nearly all warmth.

"In severe fire tests the beams have remained cold, and consequently were unaffected. When exposed to flame for a long time, the Metropolitan composition is attacked to a depth of from $\frac{3}{16}$ to

FIG. 27.

$\frac{3}{8}$ of an inch, the remainder being unaffected; and when water is thrown upon it, the mass does not fly or crack. When made thoroughly wet, as would happen from water thrown into a building during a fire, the composition is not destroyed."

In Paris a composition of plaster of Paris and broken brick, chips, etc., has been used for generations for forming ceilings between beams, so that the question of its durability is there fully settled.

The *strength* of floors made under the Metropolitan system has been accurately determined for various spans by a great number of carefully-made tests.

"The loads that so break up the composition of floors made

under this system as to require it to be replaced, vary from 1,100 to 2,000 pounds per square foot on spans of from 4 to 6 feet.

The weight of a floor finished, as shown in Fig. 26, when ready for the plaster underneath and the floor above, is about 18 pounds per square foot; and for a floor and ceiling such as is shown in Fig. 27, 24 pounds per square foot; the thickness of the floor plate is about 3¾ inches.

The proprietors of this system recommend that the floor beams be spaced about 6 feet apart, as this distance appears to give the best results with the greatest economy.

For further information concerning this system, the reader is referred to the Metropolitan Fire Proofing Co., Trenton, N. J.

There are several styles of floors constructed on the principle of the Metropolitan floor, although nearly all of the others use Portland cement concrete instead of the plaster composition. Wire lathing, expanded metal, and various shaped bars are used for the tension members. The principal advantage sought in these floors over the terra-cotta tile arches, is a reduction in the weight of the floor, thereby causing a saving in the steel construction. The floors themselves are also, as a rule, a little cheaper than the tile floors.

Another important characteristic of all floors constructed on this principle is, any settling of the arches, or filling, will tend to draw the beams or girders together, instead of pushing them apart, as is the case with tile arches; and tie rods are, therefore, unnecessary.

The strains in floors of this kind are the same as in those of a beam, the effect of the load being to pull the tension members apart at the bottom, and to crush the concrete on top. When the concrete is of the proper thickness, and of good quality, the strength of the floor will be determined by the strength of the tension members.

Several tests of beams made of Portland cement, concrete, and wire netting made by the New Jersey Wire Cloth Company, appear to show that only about one half the strength of the tension members (when of wire cloth) can be developed. In all floors constructed of concrete, plaster, or tile with steel tension members, it is of the first importance that the two materials shall be so closely united that the tension members will not be *drawn through*, or *slip* in the concrete; for the minute this occurs, the strength of the floor, *as a beam*, is destroyed.

While some of these tension member floors have been used sufficiently to fully demonstrate their strength and practicability, yet the writer believes that new arrangements or devices should be used with extreme caution and only after they have been tested and approved by experienced engineers.

Concrete and Wire Netting Floors.

Figs. 28, 29, 30, and 31, show two styles of fire-proof floors, devised by the New Jersey Wire Cloth Company, and described, together with several other applications of concrete and wire netting, in a pamphlet published by them. The segmental arch shown

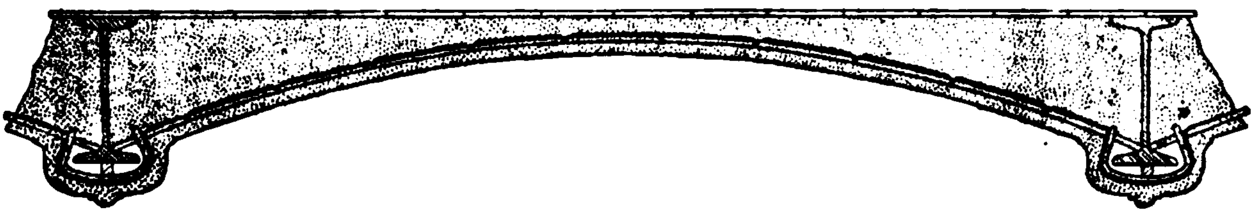


FIG. 28.

in Fig. 28 is constructed by forming a centre, made of small rods, cut the proper length to form the desired curve, and to just reach into the angles of the web and lower flange of the floor beams. These rods are inserted between the meshes of wire lathing, and the sheets, which would be three feet or more in width, are then

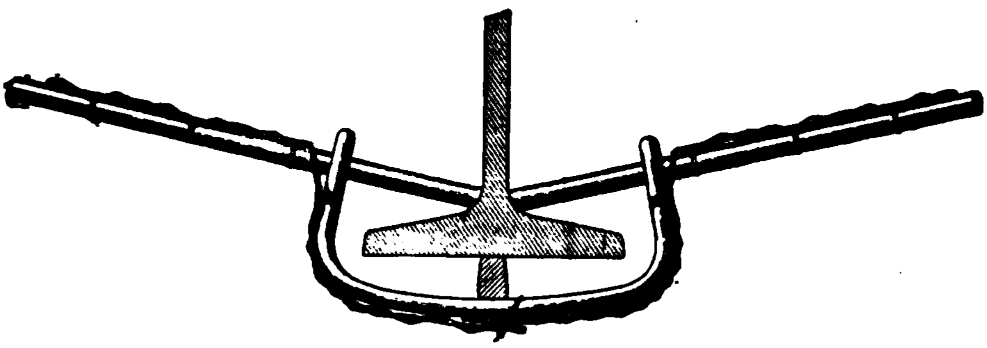


FIG. 29.

bent to the curve and sprung into place. A succession of these sheets placed side by side fill the entire space from wall to wall, and make a continuous network of iron wire and rods, upon which concrete can be spread from above without the use of any other support.

The lower flanges of the beams are covered by wire lathing attached to a succession of rods hooked over the arch rods and held in place by the wedges which are inserted between the beams and the rods.

The under side of the arches and the lathing around the beams is then plastered and finished in the usual way.

It is claimed that with this construction the strength of the arch is only limited by the ability of the beams to carry the load.

The weight of the concrete will vary from 30 to 40 pounds per square foot.

Figs. 30 and 31 show a floor construction designed on the composite beam principle.

It is claimed by the manufacturers, that a load of from 70 to 140

FIG. 30.

pounds per square foot, with a factor of safety of six, can be carried by this construction in spans of six feet between centres of beams.

The weight of the concrete, wire, and rods, for both floor and ceiling, will vary from 33 to 45 pounds per square foot.

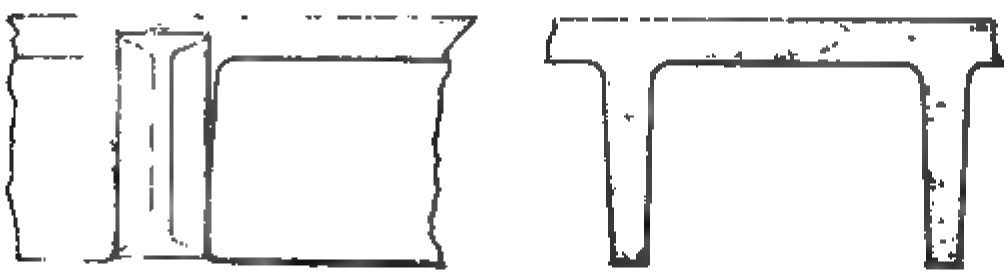


FIG. 31.

The basis of this floor construction is a series of rods hooked over the flanges of the beams, or attached to them by clips designed for that purpose. The rods are placed about twelve inches apart, and over them are spread sheets of wire lathing running parallel with them and over the top of the beams. The concrete is then spread on from above to a depth of two to three inches. No centering is required, as the cross meshes of the lathing are so close together that only enough concrete will go through to firmly anchor the wire. After the concrete is set, the under side should be plastered with cement so as to entirely embed the wire and rods.

The beams should be protected by wire lathing and plastering, and a horizontal ceiling, supported by tension rods, may be hung underneath.

Similar floors can be constructed with expanded metal lathing.

The Fawcett Ventilated Fire-Proof Floor.

This is a style of floor construction differing almost entirely from any of the floors herein described. It has been used extensively in England, and to some extent in this country.

In the construction of this fire proof floor, the special feature is a *Tubular Lintel*, or hollow tube, made of fire or red chimney-pot clay, and burned mellow.

Iron Beams (of sections to suit the spans and loads) are placed at two feet centres, and the lintels are fixed between, with their diagonals at right angles to the beams ; the end of each bay is squared by cutting (during manufacture) an ordinary lintel, parallel to the diagonal ; the piece cut off when reversed goes on the other end. Thus the ends and sides of all lintels are open next the walls. These are called "splits."

The lintels being in position, specially prepared, cement concrete is filled in between and over them, which takes a direct bearing upon the *bottom* flange of the beams, thus relieving the lintels of the floor load, which is taken by the iron and concrete, the lintels forming a permanent fire-proof centering, reducing the dead weight of the floor twenty-five per cent. and saving about half the concrete.

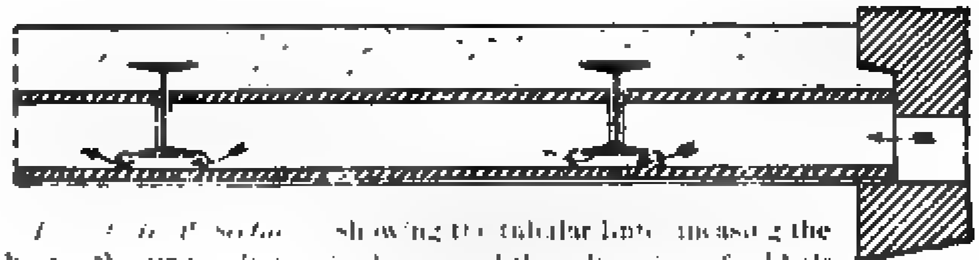
Cold Air is admitted (through air bricks in the external walls) into any of the open ends or sides of the lintels, and passes through them from bay to bay under the beams. *Note*, only two air bricks are absolutely necessary in each room, to insure a thorough current of air.

The flat bottom of the lintel completely incases the bottom flange of the beam without being in contact with it, a clear half-inch space being left for the passage of cold air.

It is claimed that the chief fire-resisting agent in this floor is not so much the terra-cotta or the concrete as the *cold air*, and that the circulation of air through the floor and around the beams will actually prevent the iron from ever getting hot at all.

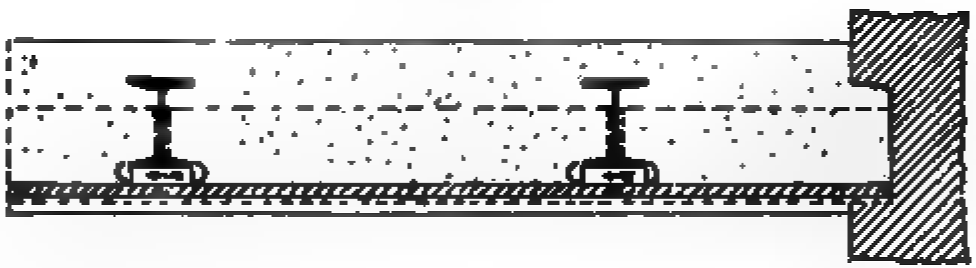
The Fawcett Company claims that their floors have never been injured by fire and water, beyond what could be repaired by replastering the ceiling and redecorating the walls. This floor needs no centering or any other support from below while in course of construction, and can be used as soon as finished. It is guaranteed to carry from 150 to 750 pounds to the square foot, according to the requirements of the building, with perfect safety.

Although the author has never seen this floor put up, it appears

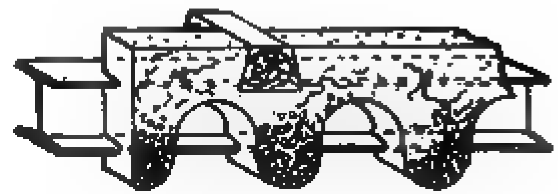


Longitudinal Section, showing the tubular lintel encasing the beam, the air passing under beam, and the admission of cold air into the end of the tubular lintel.

Lintels with concrete removed.



Longitudinal Section. Showing the concrete bearing on the bottom flange of the beam, and the cold air passage under the beam



Concrete with Lintels removed.

Transverse Section. Showing the air passage under the beams, and the admission of cold air into the side of the lintel.

Lintels fixed ready for concreting.

FIG. 31.—SECTIONS OF THE FAWCETT VENTILATED FIRE-PROOF FLOOR.

to him to be a very superior floor, although probably more expensive than the other styles herein described. It requires more constructional iron work than the systems generally in vogue in this country.

The Guastavino Tile Arch System.

Within a few years a method of constructing floors, partitions, staircases, etc., by means of thin tile cemented together so as to make one solid mass, has been introduced by R. Guastavino, of New York. The floors in this system are constructed by covering the space between the girders by a single vault, constructed of tile about 6″ × 8″, and ½ inch thick, cemented together in three or more thicknesses, depending upon the size of the vault. The thickness is generally increased at the haunches. The strength of these floor vaults, considering their thickness, appears to the author very remarkable. This method of forming floors is especially desirable where a vaulted ceiling for decorative purposes is wanted, as the vault can be made the full size of the room. The iron-work used for posts and girders must be protected as in other methods of fireproofing. The iron-work of the floors must be especially arranged for this system when it is desired to use it. As far as the author can judge from an inspection of the system, it possesses some advantages over all other present methods of construction (and, possibly, some disadvantages), and is likely to be largely used in the future. It has been employed in a number of buildings in New York and Boston, and a few other cities. The new Public Library Building in Boston has the Guastavino floor system, which is arranged so as to give a fine effect of vaulting in the ceiling.

Rules for Determining the Size of I-Beams, etc.

The method of computing the size of the iron beams used in fireproof floors is merely to determine the exact load they will have to support, and then to find the required size of beam to carry that load.

The weight of the floor itself should be determined for each particular case, as it will vary with the kind and size of tile, the amount of concrete filling, kind of flooring, etc.

The weight of the arch itself may be taken from the manufacturer's catalogue, or from the table on page 445, and to this weight should be added about 5 pounds per square foot for mortar used in setting. For each inch in depth of concrete add 8 pounds; for plastered ceiling, 8 pounds ; for hard-wood flooring, 4 pounds; for

marble floor tiles, 1 inch thick. 14 pounds. The weight of the beams may be taken at 5 pounds per square foot for 9-inch beams. and 6 pounds for 10 and 12-inch beams. Very few fire-proof floors will be found to weigh less than 75 pounds per square foot. and where marble tiles are used for the flooring the weight of the construction often reaches 95 pounds. The superimposed loads will. of course. be the same as those given on page 426. The weight to be supported by the beams will be, $w =$ distance between centers $\times$ span of beams $\times$ $(f + f')$; f representing the superimposed load, and f' the weight of the floor construction, including an allowance for the weight of the beams.

Having obtained the value of this expression, the size of beam required to carry this load may be easily obtained from the tables in Chapter XIV.

To save the labor of making these calculations in the principal classes of buildings in which fire-proof floors are used. the following tables have been computed, which may be safely relied upon.

Tables of Floor Beams.

Tables showing the size and weight of Carnegie steel beams required for different spans and spacings in different classes of buildings, using hollow tile or terra-cotta between the arches—the beams not to deflect so as to crack the plastering:

TABLE I.—FOR FLOORS IN OFFICES, HOTELS, AND APARTMENT HOUSES.

(Superimposed load, from 80 to 85 pounds per square foot.)

Span, in feet.	Distances between Centres of Beams.									
	4 feet.		4 feet 6 inches.		5 feet.		5 feet 6 inches.		6 feet.	
10	6 in.	13 lbs.	[illegible] in.	13 lbs.	6 in.	13 lbs.	6 in.	13 lbs.	6 in.	13 lbs.
11	6 "	13 "	7 "	15½ "	7 "	15½ "	7 "	15¼ "	7 "	15½ "
12	7 "	15 "	7 "	15½ "	7 "	15½ "	7 "	15½ "	7 "	15 "
13	7 "	15 "	7 "	15½ "	8 "	18 "	8 "	18 "	8 "	18 "
14	7 "	15 "	8 "	18 "	8 "	18 "	8 "	18 "	9 "	21 "
15	8 "	[illegible] "	8 "	18 "	9 "	21 "	9 "	21 "	9 "	21 "
16	9 "	21 "	9 "	21 "	9 "	21 "	9 "	21 "	10 "	25½ "
17	9	21 "	[illegible] "	21 "	10 "	25½ "	10 "	25½ "	10 "	25½ "
18	10 "	25 "	10 "	25 "	10 "	25½ "	10 "	25 "	12 "	32 "
19	10 "	25 "	10 "	25 "	10 "	25½ "	12 "	32 "	12 "	32 "
20	10 "	25 "	12 "	32 "	12 "	32 "	12 "	32 "	12 "	32 "
21	12 "	32 "	12 "	32 "	12 "	32 "	12 "	32 "	12 "	32 "
22	12 "	32 "	12 "	32 "	12 "	32 "	12 "	32 "	15 "	41 "
23	12 "	32 "	12 "	32 "	12 "	32 "	15 "	41 "	15 "	41 "
24	12 "	32 "	12 "	32 "	15 "	41 "	15 "	41 "	15 "	41 "
25	12 "	32 "	15 "	41 "	15 "	41 "	15 "	41 "	15 "	41 "

TABLE II.—FOR FLOORS IN RETAIL STORES, THEATRES, AND PUBLIC BUILDINGS.

(Superimposed load, from 125 to 130 pounds per square foot.)

Span, in feet.	DISTANCES BETWEEN CENTRES OF BEAMS.				
	4 feet.	4 feet 6 inch's.	5 feet.	5 feet 6 inch's.	6 feet.
10	6 in.—13 lbs.	6 in.—13 lbs.	7 in.—15½ lbs.	7 in.—15½ lbs.	7 in.—15½ lbs.
11	7 " —15½ "	7 " —15½ "	7 " —15½ "	8 " —18 "	8 " —18 "
12	7 " —15½ "	8 " —18 "	8 " —18 "	8 " —18 "	8 " —18 "
13	7 " —15½ "	8 " —18 "	8 " —18 "	9 " —21 "	9 " —21 "
14	8 " —18 "	9 " —21 "	9 " —21 "	9 " —21 "	10 " —25½ "
15	9 " —21 "	9 " —21 "	9 " —21 "	10 " —25½ "	10 " —25½ "
16	9 " —21 "	10 " —25½ "	10 " —25½ "	10 " —25½ "	12 " —32 "
17	10 " —25½ "	10 " —25½ "	12 " —32 "	12 " —32 "	12 " —32 "
18	10 " —25½ "	12 " —32 "	12 " —32 "	12 " —32 "	12 " —32 "
19	12 " —32 "	12 " —32 "	12 " —32 "	12 " —32 "	12 " —32 "
20	12 " —32 "	12 " —32 "	12 " —32 "	12 " —32 "	12 " —40 "

TABLE III.—FOR FLOORS IN WAREHOUSES.

(Superimposed load, from 200 to 210 pounds per square foot.)

Span, in feet.	DISTANCES BETWEEN CENTRES OF BEAMS.				
	4 feet.	4 feet 6 inch's.	5 feet.	5 feet 6 inch's.	6 feet.
10	6 in.—13 lbs.	6 in.—13 lbs.	6 in.—13 lbs.	7 in.—15½ lbs.	7 in.—15½ lbs.
11	7 " —15½ "	7 " —15½ "	7 " —15½ "	7 " —15½ "	8 " —18 "
12	7 " —15½ "	8 " —18 "	8 " —18 "	8 " 18 "	9 " —21 "
13	8 " —18 "	9 " —21 "	9 " —21 "	9 " —21 "	10 " —25½ "
14	9 " —21 "	9 " —21 "	10 " —25½ "	10 " —25½ "	10 " —25½ "
15	10 " —25½ "	10 " —25½ "	12 " —32 "	12 " —32 "	12 " 32 "
16	10 " —25½ "	12 " —32 "	12 " —32 "	12 " 32 "	12 " 40 "
17	12 " —32 "	12 " —32 "	12 " —32 "	12 " —40 "	15 " —41 "
18	12 " —32 "	15 " —41 "	15 " —41 "	15 " —41 "	15 " —41 "
19	15 " —41 "	15 " —41 "	15 " —41 "	15 " 50 "	15 " —50 "
20	15 " 41 "	15 " —41 "	15 " —50 "	15 " —50 "	15 " —50 "

It will be seen from these tables that it is more economical to space the beams farther apart, and use as short spans as the conditions of the building will permit.

For example, if we have an office floor 48 feet square, to support with iron beams and tile arches, we may either use one girder down the centre, with 12-inch beams, spaced 4 feet apart; or two girders, and 10-inch beams spaced 6 feet apart. In the former case we should require 11 beams the full width of the building, weighing

16,896 pounds, and in the latter 7 beams weighing 8,568 pounds, a saving of nearly 50 per cent. in the steel. From this, however, will have to be deducted something for extra girders and columns, but the total saving would probably equal 25 per cent. In regard to the columns, it will not make much difference in the amount of iron used, whether there are one or two rows, as the total weight to be supported is the same in either case, and if one row of girders is used the columns will be closer and heavier than if two rows are used.

Deflection of Rolled I-Beams.—The deflection of rolled iron I-beams can be computed by Formula 1, under the *Stiffness of Beams*, Chap. XVI.

According to the calculations of Mr C. L. Strobel, C.E., the beams in the foregoing tables will not deflect over one-thirtieth of an inch for every foot of span, under the load which they have been calculated to support.

Tie-rods.—Tie-rods from five-eighths to one inch in diameter are ordinarily employed to take the thrust of the brick arches, and to add to the security of the floor. These may be spaced from eight to ten times the depth of the beams apart, and the holes for them should always be punched at the centre of the depth of the beam. The formula for the diameter of the tie-rod for any floor is,

$$\text{Diameter squared} = \frac{W \times \text{span of arch, in feet}}{62832 \times \text{rise of arch, in feet}};$$

W denoting weight of floor, and superimposed load resting on the arch half-way between the tie-rods on each side.

EXAMPLE. What should be the diameter of the tie-rod to take the thrust of a 4″ brick arch, between 10″ beams, spaced 5 feet apart; the arch having a rise of 6″, and the tie-rods to be spaced 7 feet apart? The superimposed load to be taken at 100 lbs.

Ans. In this case the span = 5 feet, nearly; $W = 170 \times 5 \times 7 = 5950$; and $r = \frac{1}{2}$ foot. Then $D^2 = \frac{5 \times 5950}{62832 \times \frac{1}{2}} = .9$, or $D = 1$ inch, nearly.

Of course, where arches abut against each side of a beam, there is no need of rods to take the thrust of the arches; but it is always safer to use them, as the outside bay of the floor might be pushed off sidewise if the whole were not tied through; also, if one of the arches should fall, or break through, the rods would keep the other arches in place.

CHAPTER XXIV.

MILL CONSTRUCTION.[1]

IN this chapter it is proposed to describe the principal constructive features of what, in the Eastern States, is known as the "Mill Construction," or "Slow-burning Construction." It is a method of construction brought about largely through the influence of the factory mutual insurance companies, and especially through the efforts of Mr. William B. Whiting, whose mechanical judgment, experience, and skill as a manufacturer, have been devoted for many years to the interests of the factory mutual companies and to the improvement of factories of all kinds. Mr. Edward Atkinson, president of the Boston Manufacturers' Mutual Insurance Company, has also done a great deal towards influencing the public in favor of this mode of construction.

The *desideratum* in this mode of construction is to have a building whose outside walls shall be built of masonry (generally of brick) concentrated in piers or buttresses, with only a thin wall containing the windows between, and the floors and roof of which shall be constructed of large timbers, covered with plank of a suitable thickness; the girders being supported between the walls by wooden posts. No furring or concealed spaces are allowed, and nothing is permitted which will allow of the accumulation of dirt, the concealment of fire, or, in short, any thing that is not needed.

Mr. C. J. H. Woodbury, inspector for the factory mutual fire-insurance companies of Massachusetts, who has written a very able book on the "Fire Protection of Mills" (published by John Wiley & Sons of New York), has given such concise and clear statements of what does and what does not constitute safe construction for mills and warehouses, that with his permission we quote them *verbatim* from his work.

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[1] Cuts 1 to 6 in this chapter are taken from Woodbury's Fire Protection of Mills, and reduced, to conform to the size of the page.

"**Prevailing Features of Bad Construction of Mills and Storehouses.** — The experience of the Factory Mutuals has shown that in mill and storehouse construction, where considerations of safety, convenience, and stability are essential, the following prevalent features of bad construction should be omitted: —

"Bad roofs.

"Rafters of plank, eighteen to twenty-four inches between centres, set edgewise.

"Any roof-plank less than two inches thick (three inches preferred); any covering which is not grooved and splined.

"Any hollow space of an inch or more in a roof.

"Any and every mode of sheathing on the inside of the roof so as to leave a hollow space.

"Any and every kind of metal roof, except a tin or copper covering on plank.

"Boxed cornices of every kind.

"Bad floors containing hollow spaces or unnecessary openings.

"Thin or thick floors resting on plank set edgewise, eighteen to twenty-four inches between centres.

"All sheathing nailed to the under side of plank or timber, making a hollow floor.

"Bad finish, leaving hollow spaces, or flues.

"All inside finish which is furred off so as to leave a space between the finish and the wall.

"Wooden dados, if furred off.

"Open elevators.

"Iron doors, iron shutters.

"Any and all concealed spaces, wooden flues, or wooden ventilators of every kind, in which fire can lurk or spread, and be protected from water.

"Any and all openings from one floor to another, or from one department to another, except such as are absolutely required for the conduct of the business (all necessary openings should be protected by self-closing hatches or shutters, or by adequate wooden fire-doors covered with tin; automatic doors preferred in many places).

"**Essential Features for the Safe Construction of Mills and Storehouses.** — Solid beams, or double beams bolted near together, eight to ten feet between centres. Not to be painted, varnished, or 'filled' for at least three years after the building is finished, lest dry-rot should ensue. Ends of timbers ventilated by an inch air-space each side in the masonry.

"Roof nearly flat. Timbers laid across the tops of the walls to

project eighteen to thirty-six inches, as may be desired, serving as brackets. Plank laid to the ends of the timbers. Neither gutters nor boxed cornices of any kind. Wooden posts of suitable size, not tapered, unless when single posts turned from the trunks of trees with the heart as a centre, following the natural taper. Cores bored one and a half inches diameter; two half-inch holes transversely through the post near top and bottom for ventilation.

"Floor-planks not less than three inches thick for eight-foot bays, three and a half to four for wider bays. In some cases, beams have been placed twelve feet apart, with four-inch plank for the floor; but in such cases a careful computation of the strength should be made, based upon the load to be placed thereon, before so wide a space between beams is adopted, lest there should be excessive deflection. The better method, where the arrangement of the machinery requires such wide bays, is to alter the plan of floor-timbers. Top floor one and a quarter inch boards of Southern pine, maple, or some hard wood. The best construction requires this top floor to be laid over three-quarter inch mortar, or two thicknesses of rosin-sized sheathing-paper, certain grades of which are now made especially for this purpose.

"All rooms in which special dangers exist, such as hot drying, to be protected overhead with plastering on wire-lath, following the line of ceiling and timber, thus avoiding any cavity in the ceiling. In such rooms, the wooden posts should also be protected with tin; care being taken to leave the half-inch holes through the posts near the top and base uncovered, so that dry-rot may not take place."

Fig. 1 represents the proper construction of one bay of a three-story mill, each bay being like the others, and the building being formed of any number of such bays placed one after the other.

Such a building cannot be considered as fire-proof; but the material is in such a shape that it would not readily take fire, and would burn slowly even then. Moreover, the construction is such that any part of the building can be easily reached by a stream of water; so that a fire can be readily extinguished before it has gained much headway.

In a brick building *no granite should be used*, except for steps and underpinning, as it splits badly when exposed to heat, and is therefore unsuitable for sills or lintels, or any work liable to be exposed to any intense heat in case the building should be on fire. The best qualities of brown sandstone may be used for sills, and for other places it would be better to use brick or terra-cotta. Moulded bricks are now manufactured in a great variety of forms, and are well suited for decorative work.

The best factories and woollen mills in Massachusetts are now generally built with the beams eight feet apart from centres, and with a span of twenty-five or twenty-four feet, there being one or more rows of posts according to the size of the mill. Fig. 1 represents the section of a mill having two rows of posts.

Fig. 1.

The floor-beams are usually twelve inches by fourteen inches hard-pine timbers,[1] which rest on twenty-inch brick piers in the basement, and on wooden posts and the outside walls in the other stories. The ends which rest on the outside wall are arranged so as to have an air-space around the end of the timber, and are anchored to the wall by a cast-iron plate on which the beam rests. This plate, shown in Fig. 2, has a transverse projection on the upper surface, which fits into a groove in the bottom of the beam, and is turned down about six inches into the brickwork at the end. The brickwork for about five courses above the beam should be laid dry, and the upper edge of the end of the beam slightly rounded. In case of the possible burning of the beam, this would allow the beam to fall without throwing out the wall.

The floor on top of these beams is constructed, first, of three-inch planks, not over nine inches wide, planed both sides, and grooved on both edges, which are filled with splines of hard wood (generally hard pine) about three-fourths of an inch by an inch

[1] It has been found by experience that wood is better suited to mill construction than iron, both for beams and columns; the reason for this being that it is less rigid, and allows the machinery to run easier.

and a half. In nailing the planks, it is better to "blind nail" them, after the manner of nailing matched floors in dwelling-houses and stores; that is, driving the nails obliquely through the groove before the spline is put in: this allows the plank to shrink or swell without cracking, and without splitting the splines.

Fig. 2.

Of course, when planks are nailed in this way, each plank must be nailed before the next is put down. This takes considerable time; so that some builders lay a number of planks, wedge them up, and then drive in the splines from one end, and nail directly through the planks into the floor-timbers.

Fig. 3.

The upper flooring is generally of some hard wood, an inch and a quarter thick, merely jointed.

"The floors should be rendered water-tight by three-fourths of an inch of mortar between the upper and lower floors. The layer of mortar preserves the lumber from decay, prevents the floor from becoming soaked with oil, and is so slow burning that it is more nearly fire-proof than any other practical method of construction."[1]

Fig. 3 shows a section through such a floor as we have described. *The roof* is generally formed of ten-inch by twelve-inch hard-pine

[1] Fire Protection of Mills, p. 163.

timbers placed the same as those below; and the outside end is allowed to project over the wall from eighteen inches to two feet, forming brackets to support the eaves. These timbers are covered with two and a half or three inch spruce plank, grooved and splined the same as for the floors. The plank extend to the end of the overhanging timbers, and form the eaves to the building, no boxed cornice being allowed. If the roof is flat, as is generally the case in mills and factories, the plank should be covered with tin, gravel, or duck.

If tin is used, it should be the best "M. F." tin, painted on the under side with two coats of red-lead, and well dried before the sheets are laid.

If a gravel roof is used, it should be equal to the best quality of tar-and-gravel roofing over four thicknesses of the best roofing-felt. *Cotton duck* is gradually coming into use as a roofing material, and has for a long time been used for covering parts of vessels. It is light, durable, does not leak, and is not readily inflammable.

The material should be twelve-ounce duck, weighing sixteen ounces to the square yard, and should be thoroughly stretched, and tacked with seventeen-ounce tinned carpet-tacks, the edges being lapped about an inch. If the roof-planks are rough, or not of an even thickness, a layer of heavy roofing-paper should be laid before the duck is put down. After the duck is laid, it should be thoroughly wet, and then painted with white-lead and boiled linseed-oil before it becomes dry; which makes it water-proof. To protect from fire, give it two more coats of white-lead, and over this a coat of iron-clad paint. Instead of the four coats of white-lead and oil, the duck may be saturated with a hot application of pine-tar thinned with boiled linseed-oil. This has been found to work perfectly. The ironclad paint should be applied, whichever method is used.

If the roof is pitched, it should be covered with shingles or slate laid over three-quarters of an inch of mortar; which protects the slate from the heat, should the building take fire, and renders the roof cooler in summer, and warmer in winter, whether slate or shingles are used. Where there are no buildings near, shingles are recommended, as they are warmer than slate (thus saving in the cost of heating), and are also cooler in summer. If the shingles are painted, which is advisable, they should be dipped in paint before being laid, so as to be entirely covered on all sides with paint: otherwise, moisture will get into the shingle through the place not painted, and, being prevented from evaporating by the paint on the outside, will rot the shingle.

The columns for such a mill are usually round columns, nine inches diameter in the first story, eight in the second, and seven in

the third; these being the least diameters of the columns. If the columns are tapered, they may be half an inch less in diameter at the top, and one inch more at the bottom, making the taper on

Fig. 4.

each side of the column three-fourths of an inch. They should be of either *hard-pine* or *oak* timber, thoroughly seasoned, and should have cores bored one and a half inches in diameter, with two half-inch holes transversely through the post, near top and bottom, for ventilation and to prevent dry-rot. The columns are provided with cast-iron caps, as shown in Fig. 4, which support the ends of the floor-beams, and, where there is a vertical line of

Fig. 5.

columns, they should be provided with iron pintles, which connect the cap of one with the base-plate of the other, preventing the ends of the beams from being crushed by the weight on the columns

above. The ends of the pintles and the iron plates against which they rest should be turned true, so that the contact will be uniform. Fig. 5 represents a vertical section through the floor and the centre of the columns, and Fig. 6 shows a perspective view of a pintle with the base of the upper column coming down over the top. The brick piers in the basement supporting the columns should be capped with an iron plate twenty inches by twenty inches, an inch and three-fourths thick.

The above is the most approved method of construction now in vogue for mills, factories, and storehouses; and the dimensions given for the various parts will answer for any cotton or woollen factory where the bays are not more than eight feet long from centres. Where the bays are more than this, or the loads on the floors are greater, as may be the case in storehouses, the floor-plank and timbers should be proportioned according to the rules for strength and stiffness given in Chap. XXII., and the columns proportioned according to the rule given in Chap. XI.

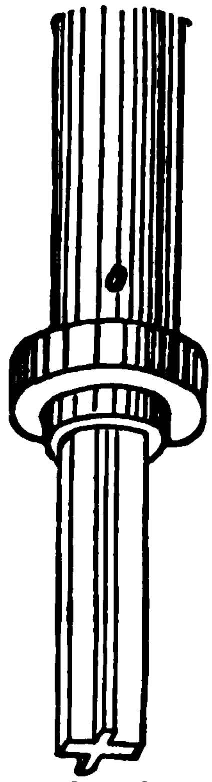


Fig. 6.

If partitions are desired in such a mill or storehouse, they should be built of two-inch tongued and grooved plank placed together on end (forming a solid partition), and plastered both sides, either on wire, or on dovetailed iron lath. Such partitions have been found to work well after a trial of twelve years, and offer effectual resistance to fire.

Mill doors and shutters should be built of two thicknesses of inch boards, covered on all sides with tin, as described in Chap. XXVI.

For a thorough description of the apparatus and appliances used for the fire protection of mills, and for a thorough discussion of the vibration of mills, the deflection of the floor-planks, and, in fact, every thing that refers to the construction and protection of mills and factories, the reader is referred to Mr. Woodbury's work on the "Fire Protection of Mills," mentioned above.

The cost per square foot of total floor area of mills and factories at the present time (1884), according to Mr. Edward Atkinson, is as follows: —

Mill with three stories for machinery, and a basement for miscellaneous purposes	75 to 80 cts.
Mill with two stories for machinery, and no basement	65 "
Mill with one story, of about one acre of floor, with basement for heating and drainage only . . .	about 85 "

The above is for the total area of floors in the building, above the basement.

not even weakened by the space left in the wall, because the anchor remains, and the crushing strength of this cast-iron box is much greater than that of the wall. No break or breach is made in the wall, and the anchor that remains, securely held, forms a space for

Fig. 9.

the easy replacement of joist. The anchor provides a perfect and secure foundation for each joist. Fire from a defective flue cannot ignite a joist end, because it is protected by a ventilated cast-iron box.

The boxes, or anchors, also have air spaces in the sides, ½ inch wide, which permit a circulation of air around the ends of the joist, effectually preventing dry rot in the ends of the timbers.

If timber is wet or unseasoned it will have a chance to dry out

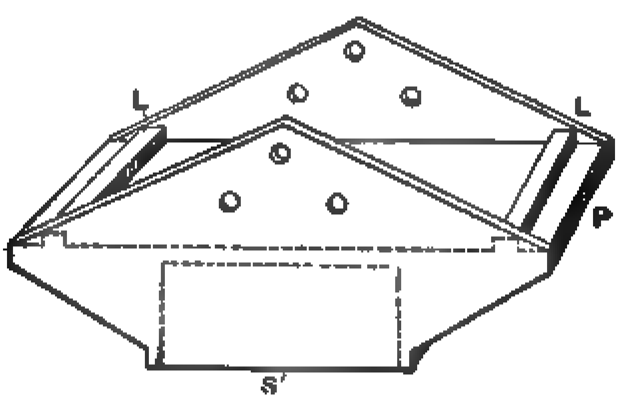


Fig. 10.

Plate *P*, 1″ to 2″ thick ; *L*, 1″ × 1″ ; socket *S*, 4″ to 5″ deep.

after it is put in the building These anchors are obviously greatly superior to the ordinary method of anchoring beams and girders to walls, and their use would in case of fire, undoubtedly save much loss by the falling of the walls, which are almost invariably

pulled down by the ordinary iron anchors. The average weight of a box like Fig. 7, for 2 × 12 joist, is 15 to 17 lbs.; of Fig. 8, from 12 to 15 lbs.

Fig. 10 shows the Goetz cap for wood posts. This cap holds all the timbers securely in place, provides ventilation about

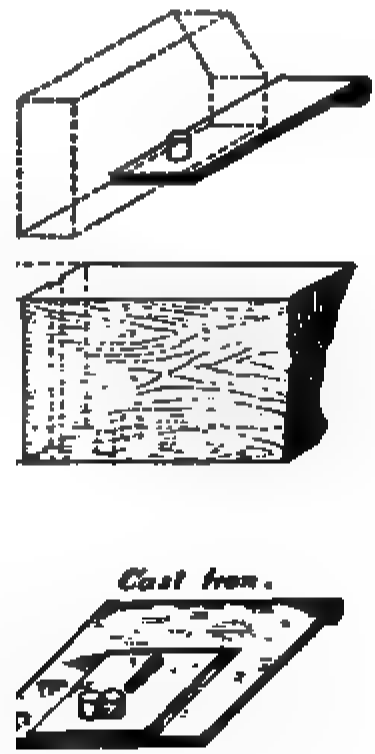


Fig. 11

the ends of the timbers, and permits the horizontal timbers to fall, in case of fire, without damage to the vertical posts.

These anchors and caps are recommended by the factory mutual insurance companies of New England, and can be made in any foundry by paying a royalty of [illegible] of a cent per pound on all that are made to the Goetz Box Anchor Company, of New Albany, Ind.

P. Davidge & Co. of Brooklyn, N. Y., have patented the anchors and caps shown in Fig. 11, and they have been used to a considerable extent. The cap differs from the Goetz cap principally in the substitution of pins for the projecting rib which holds the timbers. It is claimed that the pins do not cause the timbers to check and split, as is often the case with the rib, and it is also less work to fit the timbers to the cap. Either of these forms of caps and anchors is superior to those in common use. They must not be used, however, without a license from the patentees.

CHAPTER XXV.

MATERIALS AND METHODS OF FIRE-PROOF CONSTRUCTION FOR BUILDINGS.

The term fire-proof is applied to various kinds of buildings, sometimes correctly, but more often incorrectly.

The buildings most generally referred to by this term may be classed as follows :

1st. Those in which all the structural parts, both on the interior and exterior, are of non-combustible materials carefully protected from the action of fire by fire-resisting materials. (See also quotation from Chicago building ordinance, page 485.)

2d. Those built on the so-called "mill principle," and protected by fire-proof material.

3d. Those built in the usual manner with wooden construction, and protected by fire-proof material. Of these classes the first is the only one that is considered by experts to be absolutely impregnable to the effects of fire.

MATERIALS.

Various materials have been introduced for the purpose of making incombustible buildings, and for the purpose of fire-proof protection of other materials in structural parts of buildings, all more or less effective. Experience, however, has shown that the only materials upon which it is safe to rely are the products of clay, some concretes, and lime mortar under certain conditions. Plaster blocks have been found to be useless to withstand the effects of fire, moisture, and frost. The lime of Teil was for several years used in the manufacture of fire proof material, but to the best knowledge of the writer this has been discarded. All methods of fire-proofing by the use of exposed iron in any form are also acknowledged to be inefficient. Of all materials, burnt clay has the most numerous applications in incombustible building. It stands preëminently first as the most efficient fire-proof material in all departments of building, and especially so for interior filling of floors and partitions. For this it is used in hollow tiles of two general kinds. They are known by several different names : the one by such as porous terra-cotta, terra-cotta lumber, cellular pottery, porous til-

ing, etc.; the other by fire-clay tile, hollow pottery, hard tile, terra-cotta, dense tiling, etc. For convenience, the first is herein referred to as porous tiling, and the second as dense tiling. The terms "hollow tiling" and "fire-proof tiling" will be used when both are referred to in a general way. They will be described in their order.

Porous Tiling.—A substance formed by mixing sawdust with pure clay and submitting it to an intense heat, by the action of which the sawdust is destroyed, leaving the material light and porous, like pumice-stone. When properly made it will not crack or break from unequal heating, or from being suddenly cooled by water when in a heated condition. It can also be cut with a saw or edge tools, and nails or screws may be easily driven into it for securing interior finish, slates, tiles, etc. For the successful resistance of heat, and as a non-conductor, there is no building material equal to it. As a casing, covering, or lining for the protection of other material, it is to be preferred above every other material.

It should be manufactured from tough, plastic clays. A small percentage of fire-clay mixed in is desirable but not essential.

The proportion of sawdust should be from forty to sixty per cent., according to toughness of clay used. Care is required in manufacture that the work of mixing, drying, and burning be thoroughly done. The burning should be done in down-draught kilns by quick process. The product should be compact, tough, and hard, ringing when struck with metal. Poorly mixed, pressed, or burned tiles, or tiles from short or sandy clays, present a ragged, soft, and crumbly appearance, and are not desirable.

A fire-proof filling and protecting material should be substantial as well as incombustible. In a building made of absolutely incombustible materials it is of the first importance that the fire-proofing be able to withstand rough usage, for, in the event of fire, damage to the structural parts will be serious if the fire-proofing is dislodged, falls apart, or yields to the action of fire, or of water when a fire is in progress, or if it collapses under sudden loads, jars, or impact, although the material itself may not burn at all. In such buildings enduring qualities, both of the fire-proof material and its construction, are as vital and important as the incombustibility of the material. In the event of fire, the first danger is from the collapse of the material and not from its combustion. Experience has shown that fire-proof tiles of plastic clays, when porous, are more enduring than dense tiles, even if the dense tiles be of part or all fire-clay. Porous tiles are tough and elastic. Dense tiles are hard and brittle. The most essential requisites of a fire-proof filling and

protecting material are these: It should be tough, not easily shattered by impact; non-expansive, not easily cracked by heating or cooling; slightly elastic, yielding gradually to excessive loads, but not breaking or collapsing; compact and hard burned, but not dense; strong enough, but not of excessive crushing strength. Blocks should be light weight by being porous, but not by having thin shell and webs; should be built in between beams by such methods as bring all parts of the tiles into position to do the greatest service, whereby a structural efficiency equal to the efficiency of the material is obtained. These requirements are very fully met by properly made and properly built-in porous tiling. Shells of porous tiles should be from seven-eighths to one inch thick, and webs from three-quarters to seven-eighths, according to size of hollows.

Dense Tiling.—Next to porous tiling as a fire-resisting material must be placed dense tiling, also a product of clay. It is made into hollow tiles of much the same shape and size as porous tiling. A variety of clays are used. Most manufacturers, though not all, use more or less fire-clay, and combine with it potter's clay, plastic clays, or tough brick clays. It is very dense, and possesses high crushing strength. In outer walls exposed to weather, required to be light, it is very desirable. Some manufacturers furnish it with a semi-glazed surface for outer walls of buildings. For such use it has great durability, and effectually stops moisture. In using dense tiling for fire-proof filling, care should be taken that the tiles are free from cracks, and sound and hard burnt.

In the earlier days of fire-proof construction dense tiling seemed to supply the wants very well, but in later years the improvements in the manufacture of porous tiling have resulted in the displacement of dense tiling to a considerable extent.

Concrete.—Concrete made of Portland cement mixed with broken pieces of burnt fire-clay, broken bricks or tiles, burnt ballast or slag, and clear sand, is said to resist an intense heat successfully. It is recommended for fire-proof construction by English writers, and concrete construction has been largely used in California on account of its fire-proof qualities.

Thaddeus Hyatt, who invented the process of combining iron and concrete so as to resist transverse strains, describes a remarkably severe test by both fire and water, of concrete construction, in a work published by him, entitled, *Portland Cement Concrete Combined with Iron as a Building Material.* The concrete was heavily loaded and heated red-hot on the under side, when a stream of water was thrown against it for a period of fifteen minutes, and

the strength or durability of the concrete remained unaffected by the test.

Plaster, or Lime Mortar, when directly applied to brick or tile, will withstand the action of both fire and water; also when applied to the surface of planks and timbers by means of wire lathing, provided it fills all the space between the wire and the timber. Plaster on wire lath, applied to a ceiling on the under side of wooden joist spaced 12 or 16 inches on centres, will successfully resist any ordinary fire, but is liable to be damaged by water. Plaster blocks are not suitable as a fire-proof material. In using lime plaster for fire-proof protection, it should not contain any plaster of Paris.

Brick and Stone.—Common brick will withstand a great amount of heat without material damage, though not in so great a degree as fire-brick, porous terra-cotta, and fire-clay tile. Some sandstones do not appear to be much affected by heat, especially those containing considerable iron. Marble, limestone, and granite become completely destroyed under the action of intense heat and water, and should not be used in places where the stability of the building would be endangered by their demolition. Terra-cotta is undoubtedly the best fire-proof material for the exterior decoration of buildings.

METHODS OF CONSTRUCTION.

1. Buildings Constructed of Incombustible Materials properly Protected.—The methods of constructing fire-proof buildings have been greatly improved during the past few years, almost completely revolutionizing the old methods of building. The ideal fire-proof building should be constructed entirely of iron or steel, dressed on the outside with brick, sandstone, or terra-cotta, and protected on the inside by fire-proof materials.

The most approved method of constructing high buildings is to build the foundation on the isolated pier system, and on top of these piers place steel or wrought-iron columns extending through the entire height of the building, both on the outside walls and in the interior of the building. At each floor level iron girders are bolted to the columns, and the whole system braced by diagonal ties in the thickness of the floor. Thus is formed an iron or steel cage resting entirely on the foundation piers and which, so long as it can be kept from the action of heat and moisture, will endure forever. The outside walls are then built of brick, stone, or terra-cotta, enclosing the building and protecting its contents from the

weather. Only sufficient strength is required in this wall to withstand its own weight, and if any of it should be destroyed it would not cause the destruction of the building. The interior columns should be encased by porous terra-cotta or fire-clay tiles, finished in plaster or Keene's cement, or Portland cement if preferred, and the floors should be constructed of iron beams filled in between with tile arches, the bottom and top of the beams being carefully protected by the same material.

All partitions for dividing the various floors into rooms, corridors, etc., should be built of fire-proof partition tile, or hollow bricks, and the roof and upper ceiling should also be constructed of the same material, supported by iron-work. In such a building it is impossible for the construction of the building to be endangered by either a local fire or by a conflagration, though the inside finish may be entirely consumed. It is possible, however, to finish the building in such a way that there will be but little wood to consume, which could be easily replaced ; also, by providing fire-doors to the openings in the fire-proof partitions, any fire originating in the building can be confined to the part of the building in which it started.

DETAILS OF CONSTRUCTION.

Floors.—The various approved methods of constructing fire-proof floors have been described in Chapter XXIII.

Iron Columns.—The destruction of iron columns by incipient fires has been the common cause of the loss of vast amounts of property ever since iron columns have been used. Their destruction during fires, in buildings supposed to be fire-proof and in which incombustible materials of construction have been used, has shown the necessity for protecting them from the effects of intense heat under all circumstances. These disastrous effects have been intensified by the sudden throwing of cold water upon the heated columns, causing them to bend suddenly by contraction on the side upon which water is thrown, and consequently to break with ordinary loads. The expansion which occurs in iron columns before they have been materially weakened by heat is another element of weakness. The first result in such cases is to raise the floors or walls ; and inasmuch as the strain required to raise them is much greater than that needed to hold them, the work to be done by the columns is much greater under such circumstances.

The almost universal practice at the present day is to use wrought-iron and steel posts for the interior supports, and protect

the floors, the same material will generally be best for protecting the girders. Fig. 5c shows several ways in which this may be done.

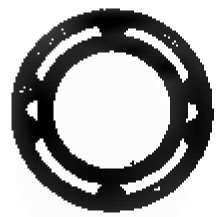


FIG. 5.—FIRE-PROOF COLUMN COVERING.

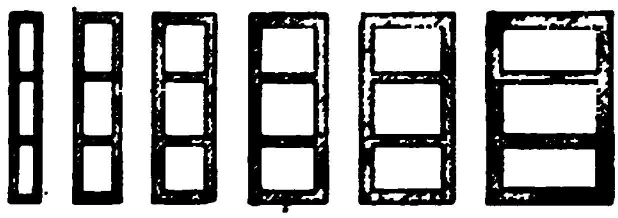
FIG. 5a.—TWO-FOOT COLUMN COVERINGS IN THE PABST BUILDING.

FIG. 5b.—SECTION OF CAST-IRON COLUMN, WITH TILE COVERING AND PLASTER.

FIG. 5c.—FIRE-PROOF GIRDER COVERING.

Partitions.

The method at present most in favor for constructing fire-proof partitions appears to be by the use of hollow blocks or tiles, of either dense or porous terra-cotta. Partitions are sometimes built by using 4-inch steel beams for studding, and fastening metal lathing on each side; but this is not as practical a partition as one made of terra-cotta blocks.



2. 3. 4. 5. 6 & 8" Partition Tile - 12" long. 12" wide

Partitions constructed of terra-cotta blocks, either dense or porous, have many valuable features other than their fire-proof qualities.

They have the greatest degree of strength combined with lightness. They are entirely vermin proof, and do not readily transmit cold, heat, or sound. When dense tile are used, courses of porous tile should be placed opposite the base or any wood mouldings, as they will receive and hold the nails while the dense tile are apt to be broken by the nails. Several styles of partition blocks are manufactured, of both dense and porous terra-cotta, some with grooved or dove tailed surfaces, and others with plain surfaces.

The weight of partition tile per square foot will average about as follows:

WEIGHT PER SQUARE FOOT OF TERRA-COTTA PARTITION BLOCKS.

Dense Terra-cotta.	Wt. per sq foot, lbs.	Porous Terra-cotta.	Wt. per sq. foot, lbs.
3 inches thick.......	13	3 inches thick......	12
4 " "	17	4 " "	17
5 " "	22	5 " "	21
6 " "	26	6 " "	26
7 " "	30	7 " "	32
8 " "	35	8 " "	38

Thin Fire-proof Partitions.—To a considerable extent in office buildings, some hotels and apartment houses, partitions are now used which finish from one and three-quarter inches to two and three-quarter inches in total thickness. There are a number of different devices and methods, all accomplishing substantially the

same results. Prominent among them are the expanded metal companies, using channel bars or flat bars and expanded metal lathing; the Lee Fire Proof Construction Company, using a core of one-inch tile, and burying Lee tension rods (similar to those used in the floors) in the plastering on each side; the Doring Fire-proofing Company, using rods, bars or channels, and burlaps; and the two-inch porous terra-cotta partition made by Henry Maurer & Son. The expanded metal system requires a scratch coat of plastering on one side, the usual brown coat work on each side, and the usual finish coat on each side—altogether, five coats for the completed partition. The Lee and Maurer systems require no scratch coat, but the usual brown coating on each side, as done with hard-setting mortar, and the finishing coats. The Doring requires a scratch coat on each side, and then the usual brown and finishing coats.

An essential thing for all thin partitions is that the plastering be of hard-setting mortar, such as Acme Cement, King's Windsor, Adamant, Rock Wall, and many others. The walls largely acquire their stiffness from the solidity of the plastering; hence the firmer and harder the plastering, the more substantial the walls.

Roofs.—For mansard roofs the most economical method of construction is by using I-beams, set 5 to 7 feet apart, and filled in between with 3-inch hollow partition tile, provision for nailing slate being made by attaching 1¼ × 2 inch wood strips to the outer face of the tile, the strips being set at the proper distances apart to receive the slate, the spaces between the strips being then plastered flush and smooth with cement mortar. In case of a severe conflagration the slate would probably be destroyed, and the wooden strips might be consumed, but the damage could go no farther. In place of partition tile porous terra-cotta bricks or blocks may be used for filling between the I-beams. For roofs where the pitch is not over 45 degrees, 3 × 3 inch T-irons, set 16 inches between centres, and filled in with slabs of porous terra-cotta, makes a very desirable roof. If slates are used they may be nailed directly into the tiles, or if it is desired to use hollow tile, strips of wood may be nailed to the tile for receiving the slate, and the spaces between the strips filled in with cement. This method may also be used for flat roofs. The best construction for flat roofs, however, is to build the roof like the floors, with tile arches between iron beams. The arches should then be covered with Portland cement, or rock asphalt, flashed around the edges with copper, and then tiled with terra-cotta tile, about 6 × 8 inches, and ¾ inch thick. This makes a durable and substantial roof, perfectly water-tight and absolutely

proof against fire. Composition, cement, and asphalt have a natural affinity for the tile, and adhere readily to it without the use of nails or fastenings. If the roof is exposed on the under side, it can be plastered and finished the same as the under side of a floor.

Trusses.—Where steel trusses are used to support the roof or several stories of a building, it is very important that they be protected not only from heat sufficient to warp them, but so that they will not expand sufficient to affect the vertical position of the columns by which they are supported.

The following description of the covering of the trusses in the new Tremont Temple, Boston, furnishes a good illustration of the way in which this should be accomplished:

"The steel girders were first placed in terra-cotta blocks, on all sides and below, these blocks being then strapped with iron all around the girders, and upon this was stretched expanded metal lathing, covered with a heavy coat of Windsor cement; over this comes iron furring, which receives a second layer of expanded metal lath, the latter, in turn, receiving the finished plaster. There is, consequently, in this arrangement for fire protection, first a dead air space, then a layer of terra-cotta, a Windsor cement covering, another dead air space, and finally the external Windsor cement."

Ceilings.—In office buildings having a flat roof, there is generally an air space, or attic, between the roof and ceiling of upper story, ranging from three to five feet in height. This space is often utilized for running pipes, wires, etc. Generally the ceiling is constructed in the same way as the floors, with the difference that lighter beams and filling are used.

It sometimes occurs that a suspended ceiling is desirable under pitch roofs, to form a finish for the upper story, and protect the roof construction. If only the weight of the ceiling itself is to be provided for, such a ceiling can be constructed at least expense by using wire or expanded-metal lathing stretched over light T's or angles, suspended from the roof construction. The angles or T's may be placed four or five feet apart, and tension rods fastened to and under them, to support the lathing; such a ceiling would weigh only about twelve pounds per square foot. Plaster boards or thin porous terra cotta blocks, placed between T-bars, also make a light ceiling, and a good ground for the plaster.

Walls.—If it is desired to further outside walls they should in no case be strapped with wood, but should be furred or lined with porous terra-cotta or fire-clay linings, as shown in Fig. 6, on which the plastering may be applied. This not only affords a protection

blocks, the same as described under Class 1. In this method of building it is also necessary to protect the upper side of the floor plank, otherwise the fire would burn through from the top. This is best done either by laying an inch of mortar between it and the upper floor, or by using hollow tile blocks laid on top of the planking, with strips between for nailing the upper flooring to.

The first method is much the cheapest, and as fire is very slow in attacking a floor, such a construction would probably resist the action of the fire as long as would the other portions of the building. The first point attacked by any fire is the ceiling of the room or story in which it originates, and every precaution must be taken to

FIG. 7—MILL CONSTRUCTION, PROTECTED BY PLASTER ON WIRE LATHING.

make the ceiling impregnable. Especial pains must be taken to see that all angles and junction of ceilings with the walls and partitions are carefully protected, so that there may be no places in which the fire may work its way through the protection back of the plastering.

Partitions.—The partitions in this class of buildings should be constructed either of hollow tile partition blocks or bricks, as in Class 1, or they may be built of 3-inch plank, tongued and grooved, and covered both sides with wire lathing from floor to ceiling, and back of the door jambs.

The Walls should either be plastered directly on the brickwork, or furred with hollow tile blocks, as previously described When carefully built, a building of this kind will be practically

Corrugated-wire Lathing consists of flat sheets of double-twist warp-lath, with corrugations $\frac{3}{8}$ of an inch deep running lengthwise at intervals of 6 inches. These sheets are made 8 × 3 feet in size, and applied directly to the under side of the floor timbers, to partitions, or brick walls, and fastened with staples. The object of the corrugation is to afford space for the mortar to clinch behind the lath, and at the same time do away with furring strips. The corrugations also strengthen the lathing. This form of lathing, however, is not as desirable as those following.

Stiffened Wire Lathing.—The Clinton stiffened wire lath has corrugated steel furring strips attached every 8 inches crosswise of the fabric, by means of metal clips. These strips constitute the furring, and the lath is applied directly to the under side of the floors or to brick walls, etc. This lath is made in 32-inch and 36-inch widths, and comes in 100 yard rolls.

The New Jersey Wire Cloth Co. also make a stiffened wire lathing by weaving into the ordinary wire cloth V-shaped strips of No. 24 sheet iron every $7\frac{1}{2}$ inches. This is an excellent lath About the only difference between it and the Clinton cloth is that the bars in the latter are attached to the cloth instead of being woven in.

Hammond's Metal Furring.—A combination of sheet-metal bearings with steel furring rods, on which ordinary wire cloth is applied, makes one of the best fire-proof ceilings. By means of this furring the plaster may be kept an inch from the bottom of the timbers, thus allowing a free circulation of the air over the ceiling. It is claimed that this is of importance in connection with fire-proofing, and is required by the building ordinance of the city of Chicago. The steel wires used for furring are so small that the mortar entirely covers them, thus securely binding the cloth and rods together, greatly stiffening the ceiling. This method may be applied to any form of construction.

Sheet-iron Lathing.—A number of styles of sheet-iron lathing have been invented and placed on the market, but they are objectionable from the fact that, in case of fire, the heat expands the iron and contracts the mortar, so that the latter becomes separated and falls off. Even without considering its fire-proof qualities, sheet-iron lathing is not desirable, as it is difficult to get a good clinch on the mortar, so as to securely hold it in place. In the wire cloth, the amount of metal in the strands of wire is so small, and it also becomes so well bedded in the mortar, that the action of intense heat does not affect it, and it has been practically demonstrated, both by actual fires in buildings and by fire tests,

elaborate decoration is to be applied, as it affords a much better surface than any other material.

The upper surface of the floor must also be protected, either by putting an inch of mortar between the under and upper floor boarding, or by filling in between the joist with fire-clay bridging tile, or by brick nogging and covering with cement mortar, on top of which the upper floor is laid. As in the previous class, especial pains must be taken to see that all corners and angles are well protected.

Roof.—If the building has a flat roof it should be protected the same as the floors, substituting for the upper floor boards, composition roofing covered with flat tiles laid in cement. For steep roofs, efficient fire-proofing becomes a difficult problem. In the opinion of the author no building, five stories high or over, should be covered with a pitch roof constructed of wood; but if such a roof is used, it can be protected for a time by covering the roof boarding with porous terra-cotta blocks, about 15 inches square and 1½ inches thick, and nailing the slate directly to them, bedding the slate in cement as it is laid; or the tile may be nailed to the rafters without boarding. For protection on the under side, if the attic space is finished, the under side of the rafters may be protected as described for ceilings; or, if the roof space is unfinished and more or less filled with trusses or other supports, a thoroughly fire-proof ceiling beneath, without any openings, would probably be as good a protection as could be obtained. The walls and partitions should be treated as in Class 2.

Complete information regarding the particular forms and sizes of the various fire-proof blocks manufactured may be obtained by addressing *The Raritan Hollow and Porous Brick Co.*, of New York City; *The Wight Fire-proofing Co.*, of Chicago or New York; *The Pioneer Fire-proof Construction Co.*, of Chicago; *Henry Maurer & Son*, New York City; *The Lee Fire-proof Construction Co.*, New York; and *The Staten Island Terra Cotta Lumber Co.*, New York City.

Details, Finish, etc.

After the constructive portions of the building are completed and the building is plastered, there are yet many details to be arranged, so as to afford the least possible material for a fire, and also combine strength, durability, and often elegance.

Stairs.—The most important of these are the stairs, which, owing to the necessity of their being located in a sort of well or shaft, are always fiercely attacked by a fire. To construct a thoroughly fire-proof stair is rather a difficult undertaking. Many architects content themselves by merely making the strings and risers of wrought

or cast-iron, and the treads of slate, marble, or wood. Such stairs are undoubtedly far better than the ordinary wooden stairs, but they are merely incombustible. In building such stairs wrought-iron strings should be used with slate treads and iron risers. Twelve-inch channel bars make excellent strings, turning the flanges out, and bolting the risers to the stem as shown in Fig. 9.

The best stairs for a fire-proof building are those built of brick or Portland cement concrete, with at least one end supported by a brick wall. If concrete stairs are constructed they should be built square and solid—that is, having the same shape on the bottom as on the top. If the stairs are built between two brick walls, as should always be the case in a theatre, they will have sufficient strength by extending them 4 inches into the brick wall. If only one end is supported by a wall, the other end can be supported by wrought-iron strings built into the concrete.

Fig. 10 shows two sections of a brick stairway. Stairways simi-

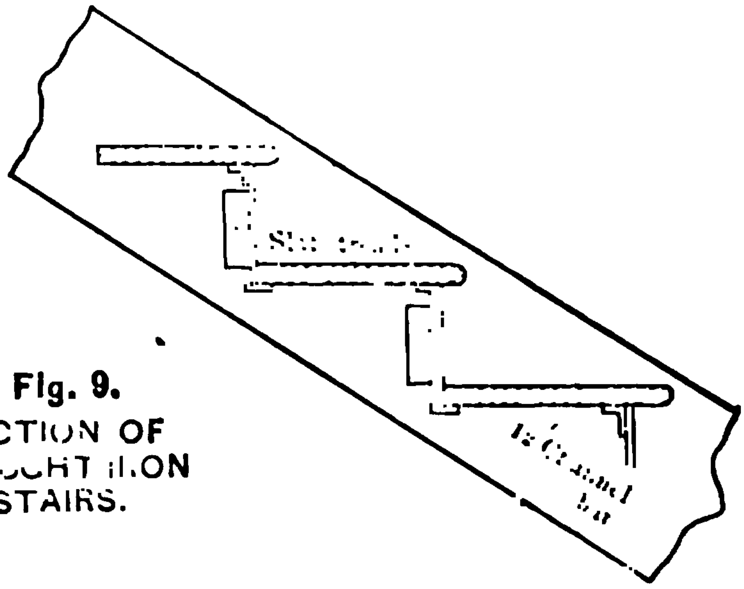


Fig. 9.
SECTION OF
WROUGHT IRON
STAIRS.

lar [illegible] in use in the new Pension Building at Washington [illegible] may be considered as absolutely fire-proof. Next to [illegible] cement stairs, the author would recommend stairs constructed with [illegible] wrought or cast iron strings, protected on the under [illegible] porous terra-cotta tile, and with slate [illegible] with tile or plaster on boards [illegible] strings. Iron treads have been found undesir- [illegible] Fire is very slow to attack a smooth [illegible] if there is no air space behind it, so that [illegible] the above would be far better than [illegible]

[illegible] section through such a stairs. The strings [illegible] with as [illegible] ornamentation [illegible]

Granite Stairs.—In many of the Government buildings the

stairs are constructed all of granite, a section through the steps being like that shown in Fig. 12. One end of the steps is built into a wall, and the other depends upon the shape of the steps for support

Granite and most other natural stones, however, are readily destroyed by the action of fire and water, so that such stairs can in no way be considered as fire-proof.

As to the stair railing, if brick stairs are used, some form of brick

... next ... railing.

... and also ... in case of ...

... should be the ...

... When ... pockets in the ...

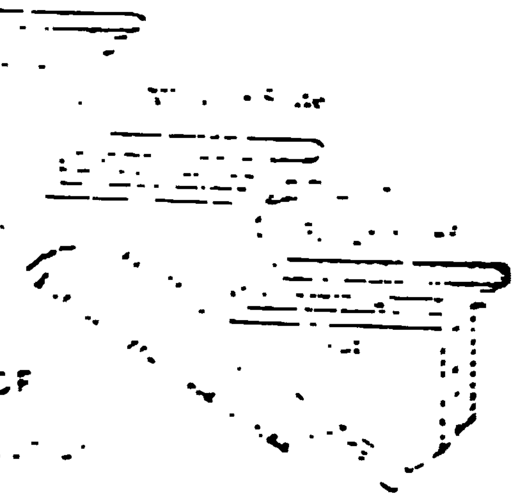


Fig. 11.
FIRE PROOF
STAIRS

... can generally ...

... exception of the floor ... cement. By ... and it can ... The window ...

... artificial ... is perfectly fireproof ... and is very dur...

Ventilation and Hot-air Flues. These should in all ... be taken that ... them.

... this class ... be heated either by steam or hot water ... the best methods for heating offices is described

in the article on Steam-Heating, under Direct-Indirect Radiation. If this method is employed, no hot-air flues will be needed, and it will only be necessary to provide for ventilation flues.

In running iron and lead pipes, etc., in the walls and partitions, they should run in channels in the brickwork, and be covered with

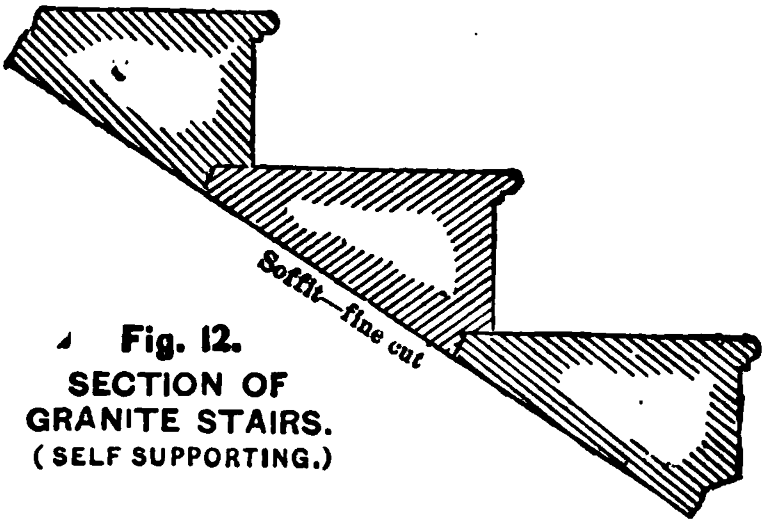


Fig. 12.
SECTION OF
GRANITE STAIRS.
(SELF SUPPORTING.)

sheets of boiler iron about three-sixteenths of an inch thick, put up with screws, in an iron frame fastened to the brickwork.

This can be painted as desired, and afford ready access to the pipes.

No pipes should be carried in a wall or partition where they are not accessible.

In finishing around elevator doorways, etc., where considerable ornamentation is required, cast-iron, painted in color, can be used with good results. Where there is no combustible material, there can of course be no fire.

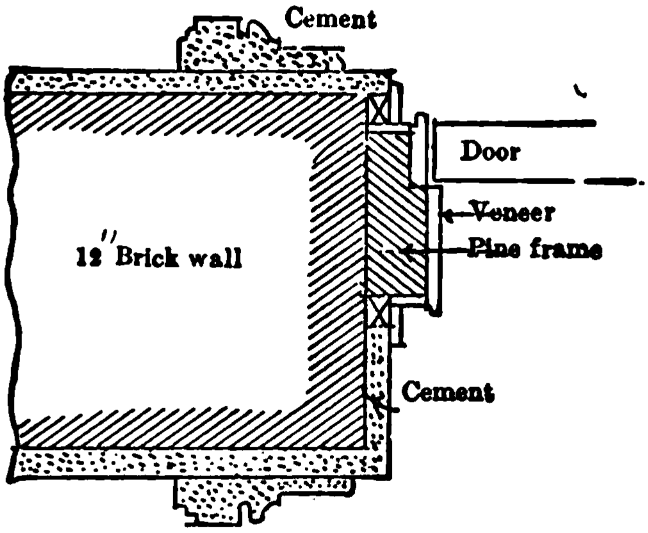


Fig. 13.
SECTION THRO' DOOR JAMB

Stand-pipes.—A very important adjunct to every fire-proof building is a stand-pipe of 2-inch wrought iron, connected with the street main and running up above the roof (if flat), and provided on each floor with suitable valves, hose, etc., ready for instant use.

thousand square feet, without special permission, based upon unusual and satisfactory precautions.

6. That every building to be erected, which shall be three stories high or more, except dwelling houses for one family, and which shall cover an area of more than twenty-five hundred square feet, should be provided with incombustible staircases, enclosed in brick walls, at the rate of one such staircase for every twenty-five hundred square feet in area of ground covered.

7. That wooden buildings, erected within eighteen inches of the line between the lot on which they stand and the adjoining property, should have the walls next the adjoining property of brick; or when built within three feet of each other, should have the walls next to each other built of brick.

8. That the owner of an estate in which a fire originates should be responsible for damage caused by the spread of the fire beyond his own estate, if it should be proved that in his building the foregoing provisions were not complied with. A certificate from the Inspector of Buildings shall be considered sufficient evidence of such compliance, if the building shall not have been altered since the certificate was issued.

In addition to these general propositions, another series of suggestions was adopted, providing for proper fire-stops between the stringers in wooden stairs, and between all studdings and furrings, in the thickness of the floors, and for six inches above; for carrying brick party-walls, and outside walls adjoining neighboring property, above the roof, and for anchoring wooden floor-beams to brick walls in such a way as to prevent the overthrowing of the walls in case the beams should be burned off and fall.

Chicago Definition of Fire-proof Construction.

"The term 'Fire-proof Construction' shall apply to all buildings in which all parts that carry weights or resist strains, and also all stairs and all elevator enclosures and their contents, are made entirely of incombustible material, and in which all metallic structural members are protected against the effects of fire by coverings of a material which must be entirely incombustible and a slow heat conductor. The materials which shall be considered as fulfilling the conditions of fire-proof covering are: First, brick; second, hollow tiles of burnt clay applied to the metal in a bed of mortar and constructed in such manner that there shall be two air spaces of at least three-fourths of an inch each by the width of the metal surface to be covered, within the said clay covering; third, porous terra-cotta which shall be at least two inches thick, and shall also be applied direct to the metal in a bed of mortar; fourth, two layers of plastering on metal lath."

CHAPTER XXVI.

WOODEN ROOF-TRUSSES, WITH DETAILS.[1]

WHENEVER it is required to roof a hall, room, or building, where the clear span is more than twenty-five feet, the roof should be supported by a truss of some form. The various forms of trusses used for this purpose have certain features and principles in common, differing from those in bridge and floor trusses, which have

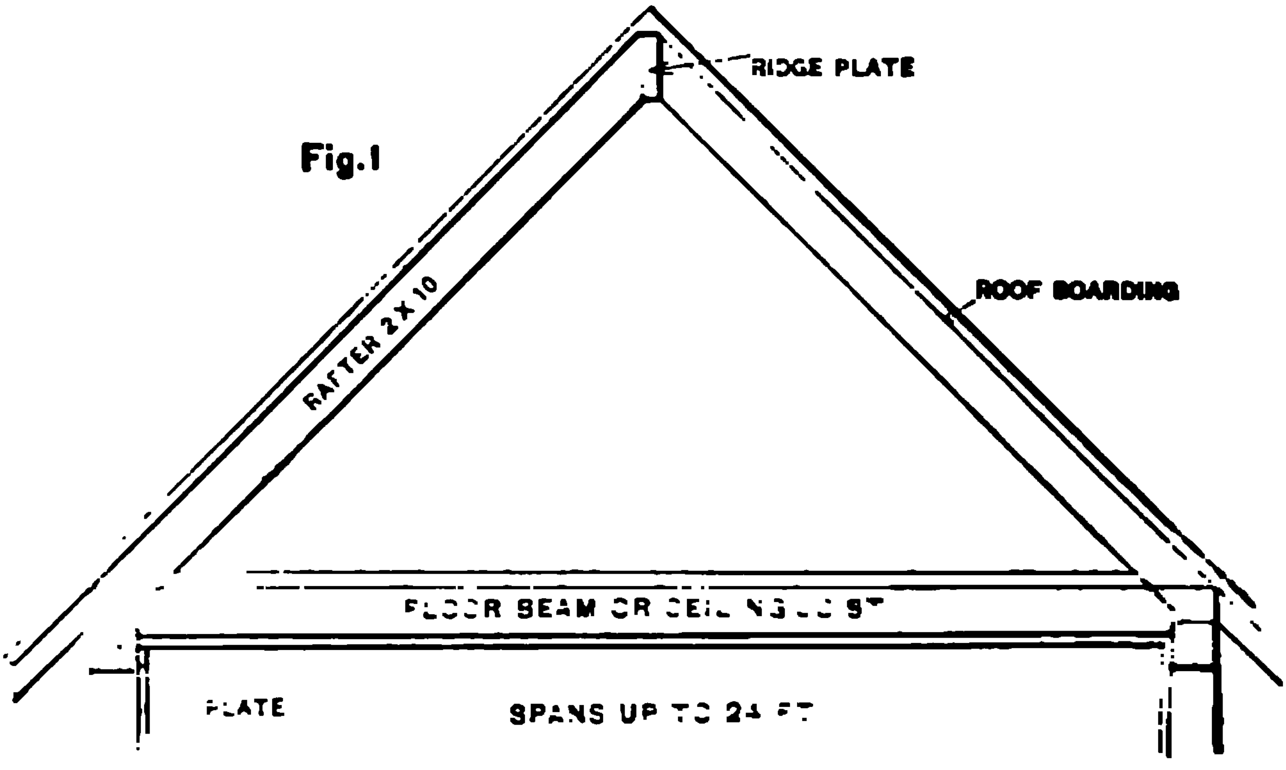


Fig. 1

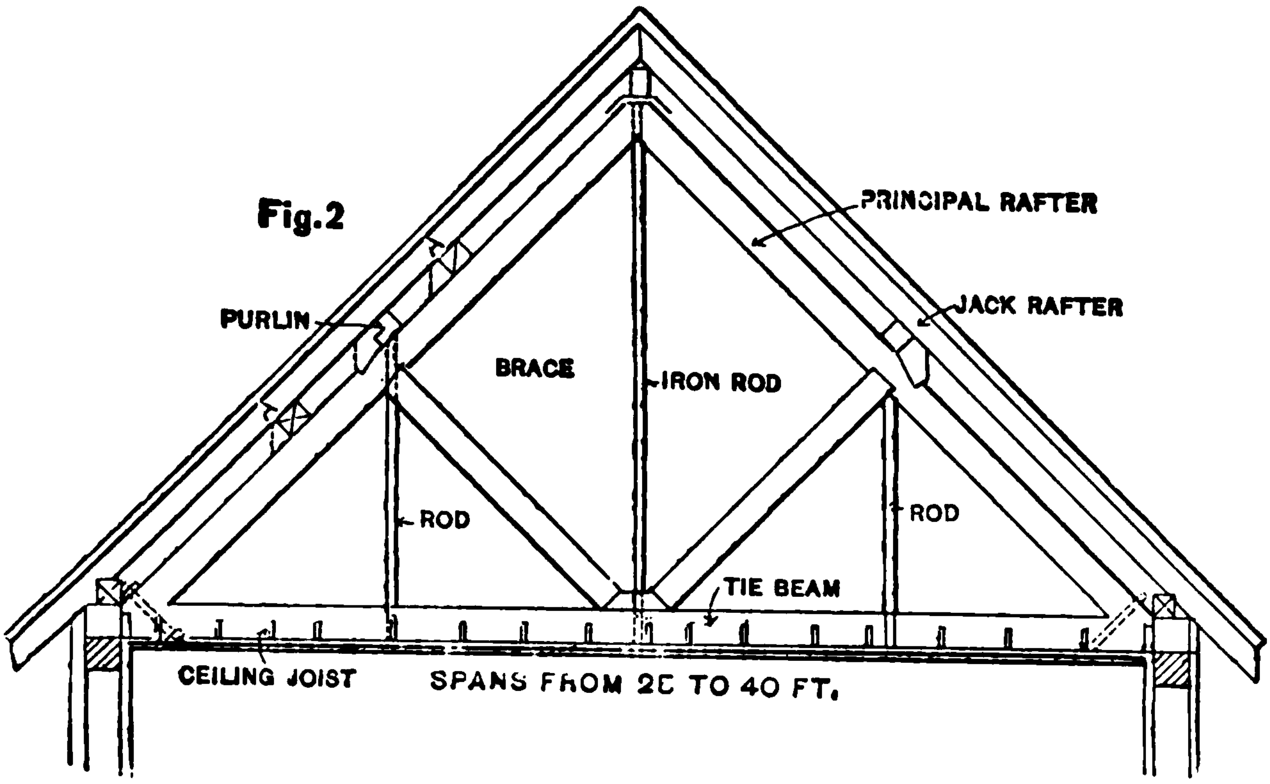
led to grouping them in one class, called "roof-trusses." Nearly all roof-trusses in churches, and halls of like character, and the larger proportion of trusses used in all kinds of buildings, are constructed principally of wood, with only iron tie-rods and bolts; and, as wooden trusses are of more interest to the architect and builder than iron trusses, they have been more completely described, and a greater variety of forms are given than for iron

[1] The details of the various trusses shown are all drawn slightly out of scale, in order to show how they are joined together. The trusses thus look heavy in proportion to the span; and it should be borne in mind that the figures are not to show the size of the timbers, but the relation of the various parts.

roof-trusses, which are discussed in another chapter. In the Northern States and Canada, where there are often heavy snow-storms, experience has taught that the best form of roof for a building, except, perhaps, in large cities, is the Λ, or pitch roof.

The inclinations of the roof may vary from twenty-six degrees, or six inches to the foot, to sixty degrees, or twenty-one inches to the foot, but should not be less than six inches to the foot for roofs covered with slate or shingles. For roofs covered with composition roofing, tin, or copper, the inclination may be as little as five-eighths of an inch to the foot.

Fig.2



The simplest form of pitch roof is that shown in Fig. 1. It consists simply of two by ten or two by twelve inch rafters, supported at their lower ends by the wall-plate, and holding themselves up at the top by their own stiffness and strength. A piece of board, called the "ridge-plate," is generally placed between the upper ends of the rafters, and the rafters are nailed to it. In some localities this ridge-piece is not used, but the upper ends of each pair of rafters are held together by a piece of board nailed to the side of the rafters before they are raised.

The walls of the building are prevented from being pushed outward by the floor or ceiling beams, which are nailed to the plate. The rafters are placed about two feet, or twenty inches, on centres, and the boarding is nailed directly on the rafters. The horizontal joists support the attic-floor and the ceiling of the room below. Such a roof can only be used, however, when the distance between the wall-plates is not more than twenty-four feet; for with a greater span the rafters, unless made extremely heavy, will sag very considerably.

King Post Truss.—Whenever we wish to roof a building in which the wall-plates are more than twenty-four feet apart, we must adopt some method for supporting the rafters at the centre. The method generally employed (shown in Fig. 2) is to use trusses like that shown in the figure, spaced about twelve feet apart in the length of the building, and on these place large beams, called "purlins," which support the roof, or jack-rafters. As the distance from one purlin to the next is not generally more than six or eight feet, the jack-rafters may be made as small as two inches by six inches. When the span of the truss is more than thirty-four feet, two purlins might be placed on each side of the truss, or at A and A. It is always best, however, to place the purlins only over the end of a brace, or at a joint, when it can be so arranged. The ceiling of the room covered by the roof is framed with light joists supported by

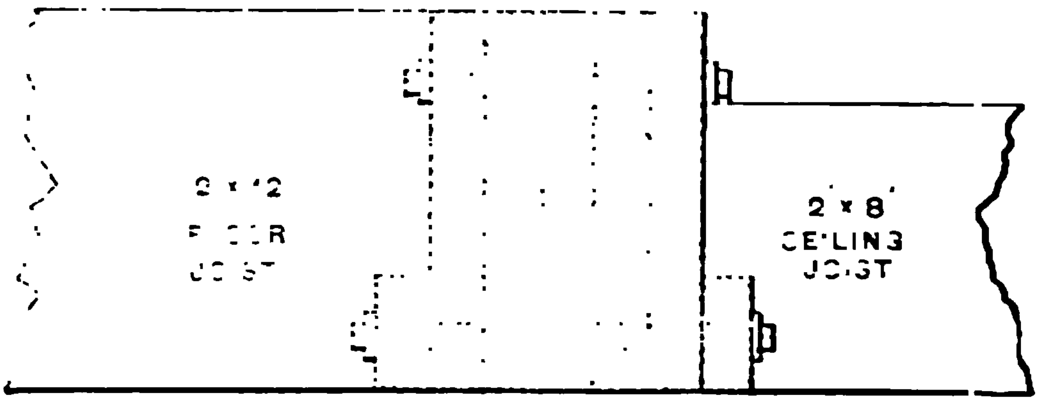


Fig. 3

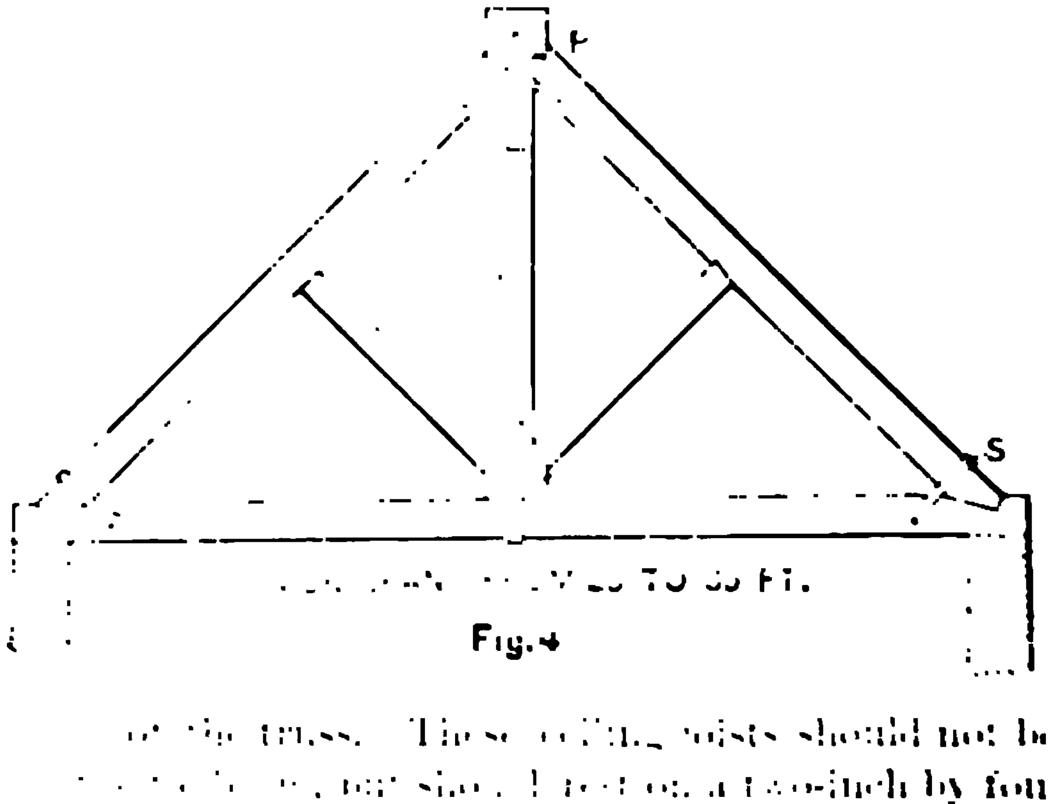


Fig. 4

[illegible] of the truss. These ceiling joists should not be mor[e] [illegible] on a two-inch by four-inch [illegible] as shown in Fig. 4.

W[illegible] the truss exceeds thirty-five feet, it is difficult to [illegible] for the tie-beam without splicing, and [illegible] one of the best methods of building up the tie-

beam is to make it of two-inch plank bolted together, the pieces breaking joint, so that no two joints shall be opposite each other. This form of truss is very rarely used where the timbers may be seen from the room below, and they are therefore generally left rough. If they were to be planed, and made a part of the finish of the room below, it would be necessary to use solid tie-beams spliced together, or else build the truss of hard pine, of which wood, timbers may be obtained fifty or sixty feet long. The form of truss shown in Fig. 2 is the modern form of the old king post truss, shown in Fig. 4, which was made wholly of wood, excepting the iron straps used to connect the pieces at the joints.

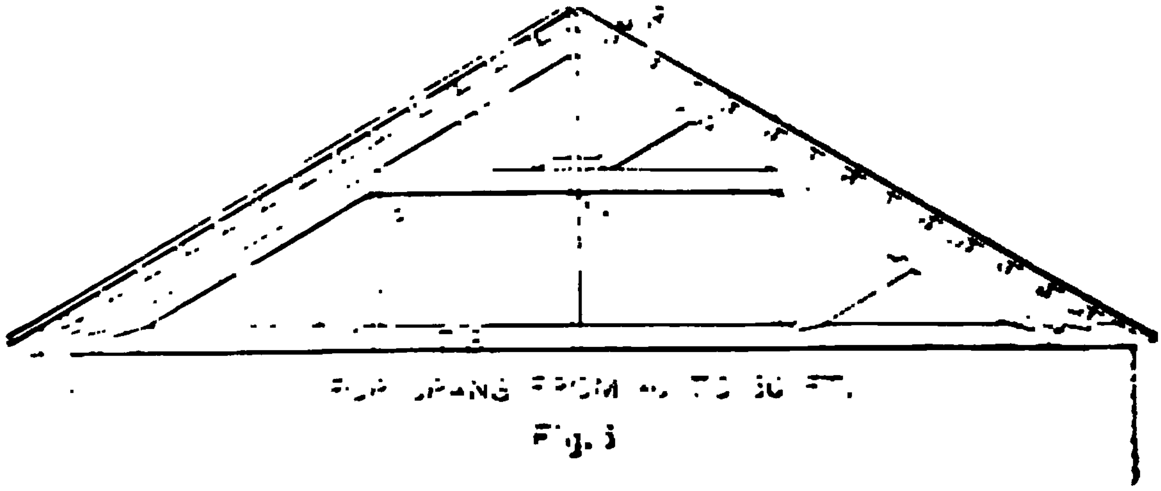
Queen Post Truss. — When the span to be roofed is between thirty-five and forty-five feet, a truss such as is shown in Fig. 5 is preferable, for several reasons, to the king post truss.

It consists of a horizontal straining-beam, separating the upper ends of the principal rafters, and a rod at each end of the straining-beam, leaving a large space in the centre of the beam clear. This is a great advantage in many cases where it is desired to utilize the attic for rooms.

This form of truss should not be used for a span of over forty feet. For spans from forty feet to fifty feet, another form of the same truss, shown in Fig. 6, should be used. This is a very strong form of truss, and leaves considerable clear space in the centre. In this truss the principal rafter should be made of two pieces, — one running to the top, the other only to the straining-beam. This gives the greatest economy in construc-

[illegible] and [illegible] forming a proper joint at B. It should be [illegible] that the strength of a truss depends largely upon [illegible] are joined together, and that a truss- [illegible]

[illegible] buildings in London [illegible] and has [illegible]



FOR SPANS FROM 40 TO 50 FT.

Fig. 5

[illegible] the same as [illegible] dimensions of the truss pieces [illegible]

[illegible] Massachusetts [illegible]

[illegible]

[illegible] rafter is secured [illegible] uncommon.

l an enlarged detail of it is shown in Fig. 10. This truss is m the Museum of Fine Arts, St. Louis, Mo., Messrs. Peabody & earns, architects, Boston, Mass.

For spans of from forty to eighty feet, a truss such as is shown in ;. 11 is one of the best forms to adopt, where a pitch roof is lired.

The struts should be largest towards the centre, and the tie-rods o.

The main rafter, on the contrary, and the tie-beam, have the atest strain at the joint A. Figs. 12 and 13 show details of nts A and C.

The trusses which have thus far been given are the simplest forms of modern trusses for spanning openings up to sixty or seventy-five feet in width, or even greater, where it is desired to have a pitch roof.

At the present day, however, flat roofs are very extensively used; and, when it is desired to carry a flat roof, a different form of truss will be found more economical.

ROOF TRUSS WITH FLOOR SUSPENDED FROM IT.

Fig. 9

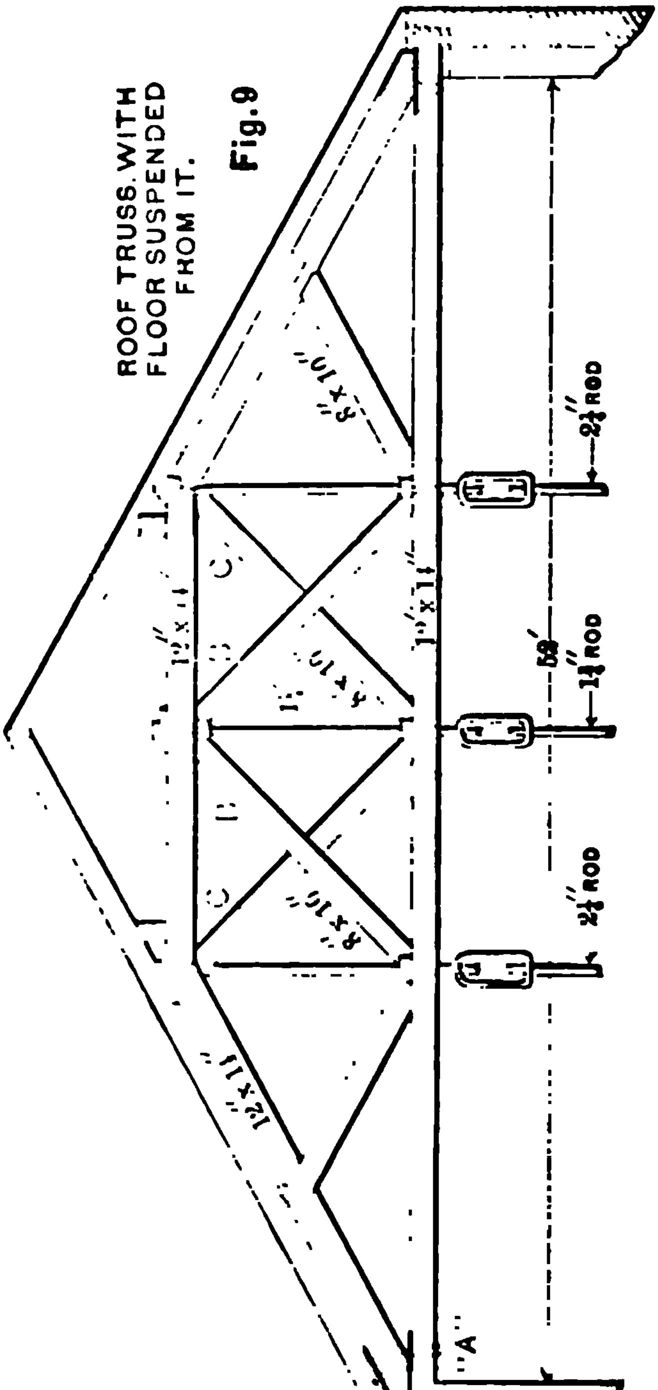
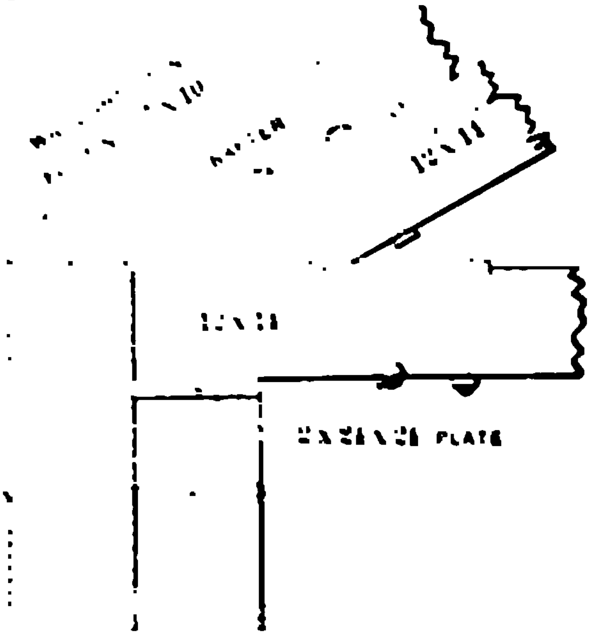


Fig. 10

DETAIL OF JOINT "A" FIG. 9



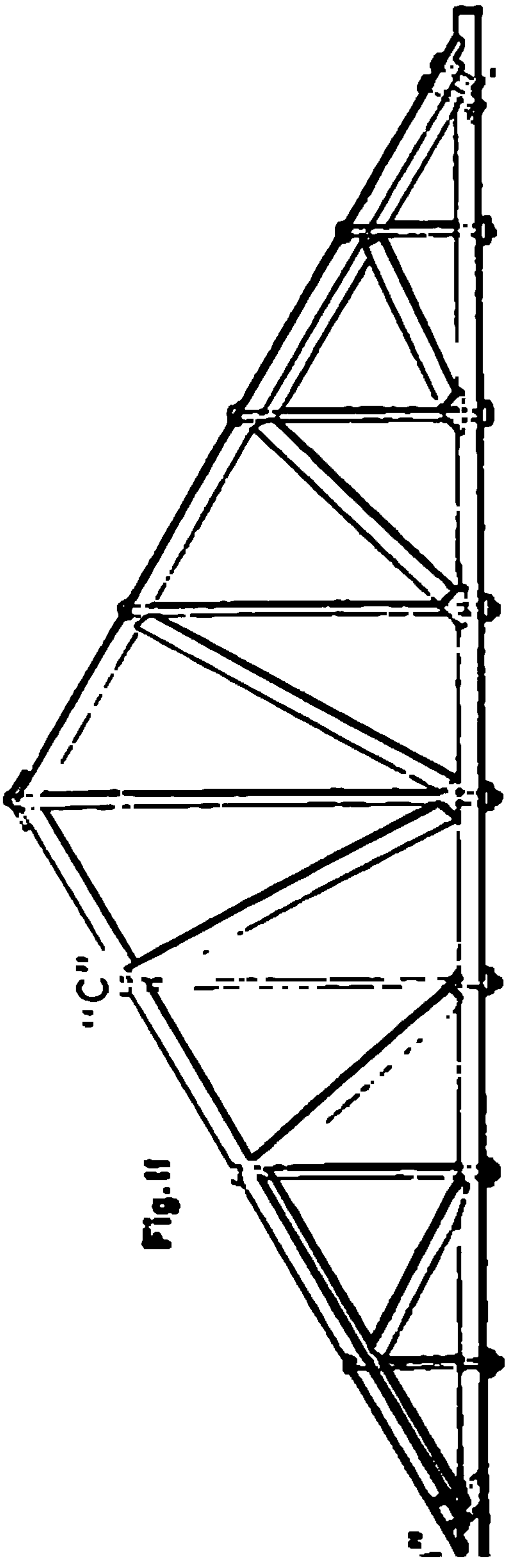


Fig. 11

SUITABLE FOR SPANS UP TO 60 FT.

The form of truss generally employed for flat roofs is that shown in Figs. 14 and 15. This truss may be adapted to any span from

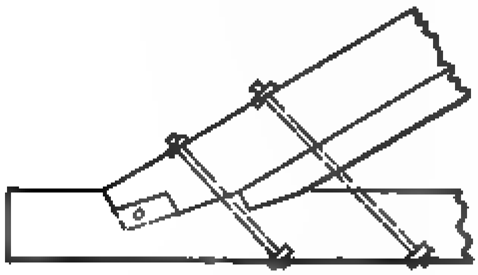


Fig. 12
DETAIL OF JOINT "A," FIG. 11.

twenty to one hundred feet, by simply changing the height of the truss and the number of braces, and proportioning the various parts to the strains which they carry. The height of the truss between the centres of the chords ought

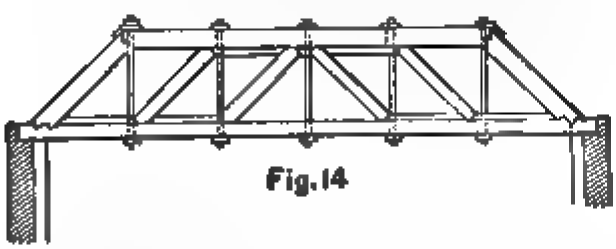


Fig. 14

not to be less than one-eighth of the span, and, if possible, should be made one-seventh, as the higher the truss, the less will be the strain on the chords.[1]

It should be noticed, that in this truss the braces are inclined in the opposite direction to that in which they are placed in the

[1] The two horizontal pieces are called the "chords," the top one, the upper chord, which is always in compression; and the bottom one, the lower chord, which is always in tension.

trusses previously shown. The distances between the vertical rods should be so arranged that the braces shall not make an angle of more than forty-five degrees with a horizontal line.

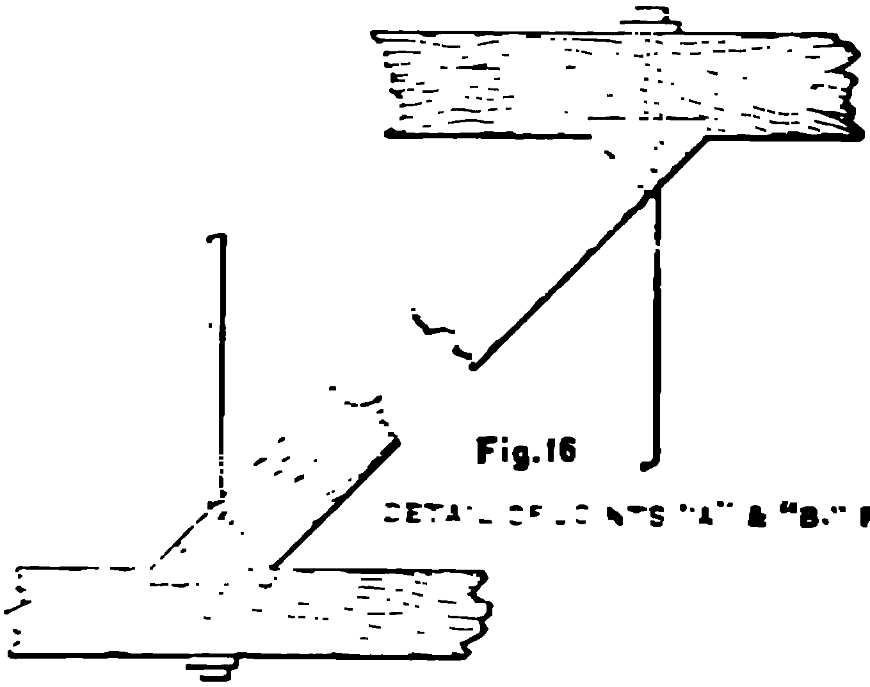


Fig. 16

DETAIL OF JOINTS "A" & "B." FIG. 15.

Fig. 16 shows the best method of forming the joints, *A*, *A*, *A*, *B*, *B*, *B*, etc. Fig. 17, although not very frequently used in roof-trusses. For spans over forty feet, the tie-beam should be made up of plank bolted together, as shown in Fig. 3, unless it is possible to have the tie-beam in one piece. This is a good form of truss for theatres, and large halls where there is a horizontal ceiling.

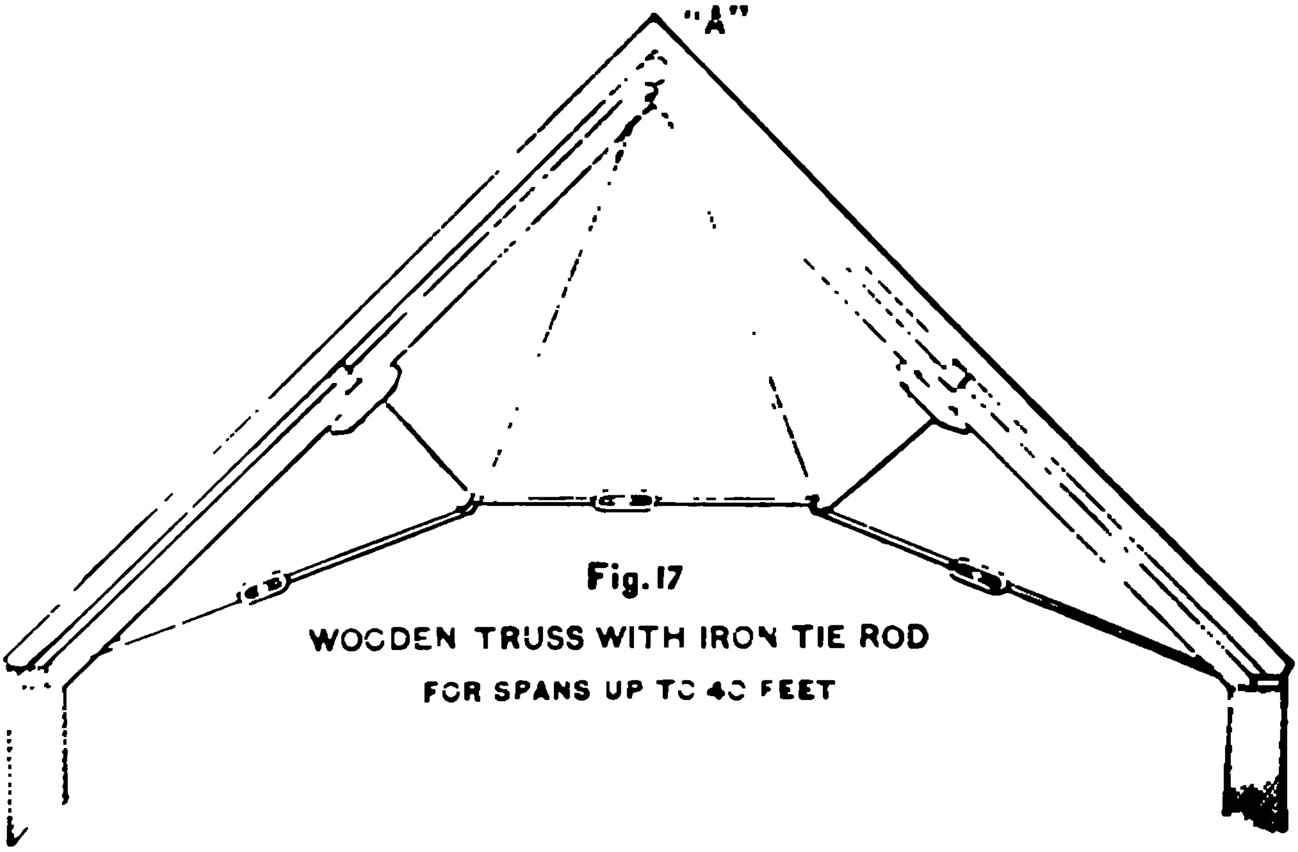


Fig. 17

WOODEN TRUSS WITH IRON TIE ROD

FOR SPANS UP TO 40 FEET

Counter-Braces.—If it is desired to load the truss at any point other than the centre with a concentrated load,—as, for instance, suspending a gallery by means of rods from the roof-

trusses,—the truss should have additional braces, called "counter-braces," slanting in the opposite direction to the braces shown.

These counter-braces need only be used when the truss is unsymmetrically loaded.

Wooden Trusses with Iron Ties.—In all trusses where the tie-beam of the truss is not horizontal, but higher in the centre than at the ends, it is better to substitute an iron tie for the wooden tie-beam.

Fig. 17 shows a form of truss very well suited for the roofs of carriage-houses, stables, or any place where it is desired to have considerable height in the centre of the room, and a ceiling is not desired.

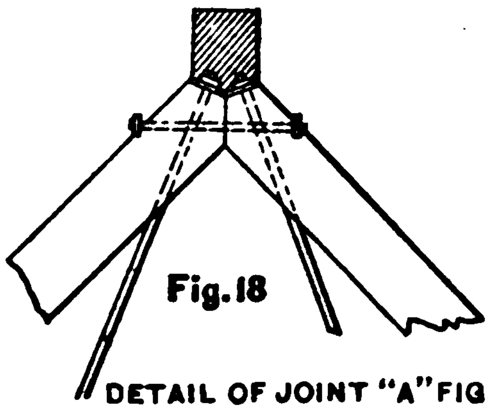


Fig. 18

DETAIL OF JOINT "A" FIG. 17

The horizontal iron rod is fastened to the two struts at their ends, and the other two rods are fastened only at their ends, and merely run over the end of a strut in a groove. The iron rods are tightened by means of the turn-buckles shown on the drawing. Fig. 18 shows a detail of the upper joint *A*. A better way of making the joint would be to have an iron box cast to receive the end of the rafters, and fasten the ends of the tie.

Arched Trusses with Iron Tie-Rods.—For buildings where it is desired to have the trusses and roof-timbers show, with

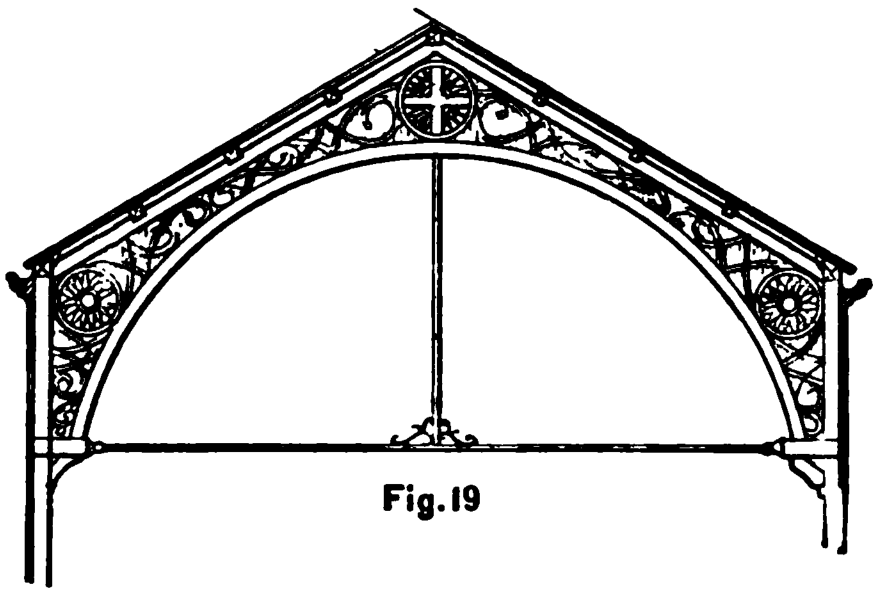


Fig. 19

no ceiling but that formed by the roof, a very pretty and graceful form of truss is obtained by the use of arched ribs, either for the principal chords of the truss, or for braces. In such trusses an iron tie-rod adds to the grace and apparent lightness of the truss, and may be very conveniently used. Fig. 19 shows a form of truss used to support the roof of the Metropolitan Concert Hall, New-York City, George B. Post, architect. The span of the truss in

the purlins and rafters, and only carries the load directly
·ch. It does not assist the truss in any way in carrying

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rovide for the
these large arches without having rods showing in the
d the method adopted is very ingenious. Opposite the
ls of the iron posts which receive the arched ribs are oak

CARL PFEIFFER, ARCHITECT. Fig. 22 G. BOUSCAREN, C.E.

struts, which are held in place by iron tie-bars and heavy iron beams, which together form a horizontal truss at each end. These two trusses are prevented from being pushed out by two three-inch by one-inch tie-bars in each side wall shown in the plan (Fig. 23).

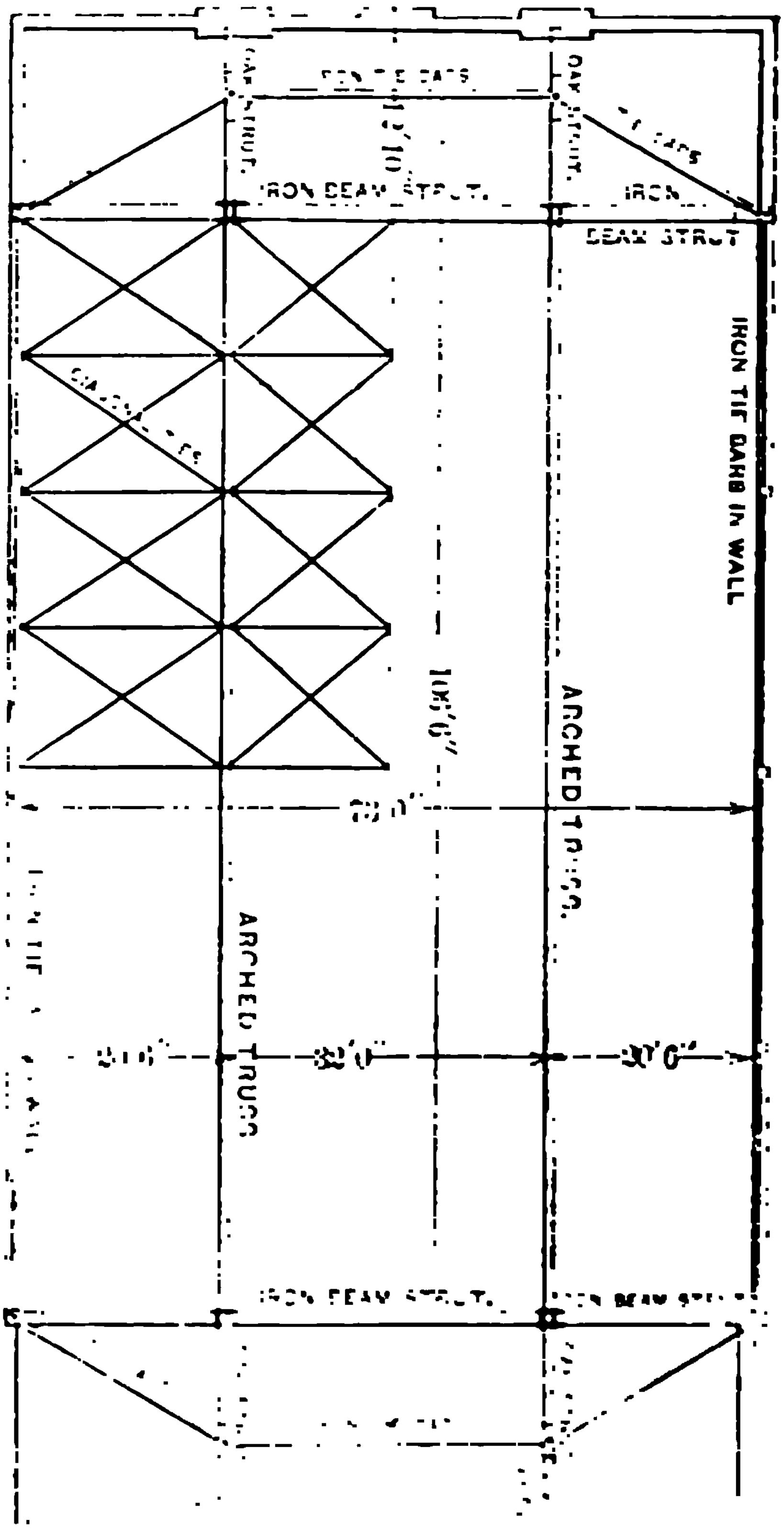


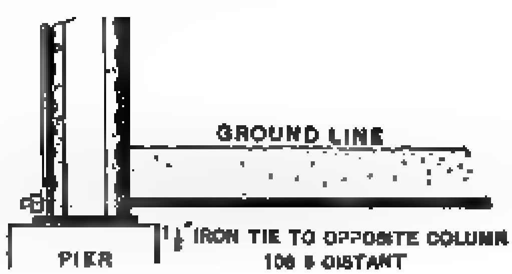
Fig. 23
PLAN SHOWING PRINCIPLE OF CONSTRUCTION.

The bottoms of the two iron posts are tied together by iron rods running under the floor the whole length of the room. Altogether [illegible] two bars three inches by [illegible] which would be [illegible] bars three inches and three-fourths by one [illegible] sections of the ribs, uprights, and braces, are shown in Fig. 22. It should be noticed that the uprights act both

nts and ties, by having iron rods through their centre holding ro ribs together.

. 24 shows a detail, or enlarged view, of the iron skewback ost at each end of the truss shown in Fig. 22.

. 25 shows the method adopted for supporting the roof and y of the City Armory at Cleveland, O.



en-Timber Trusses. — One of the principal character- of the Gothic style of architecture is that of making the ural portions of the building ornamental, and exposing the construction of an edifice to view; and, as the pointed s and steep roofs were developed, the roof-truss became an tant feature in the ornamentation of the interior of the c churches.

se trusses were built almost entirely of wood, and generally ry heavy timbers, to give the appearance of great strength. f the simplest forms of these trusses is shown in Fig. 26. As e seen in the figure, the truss is really not much more than a

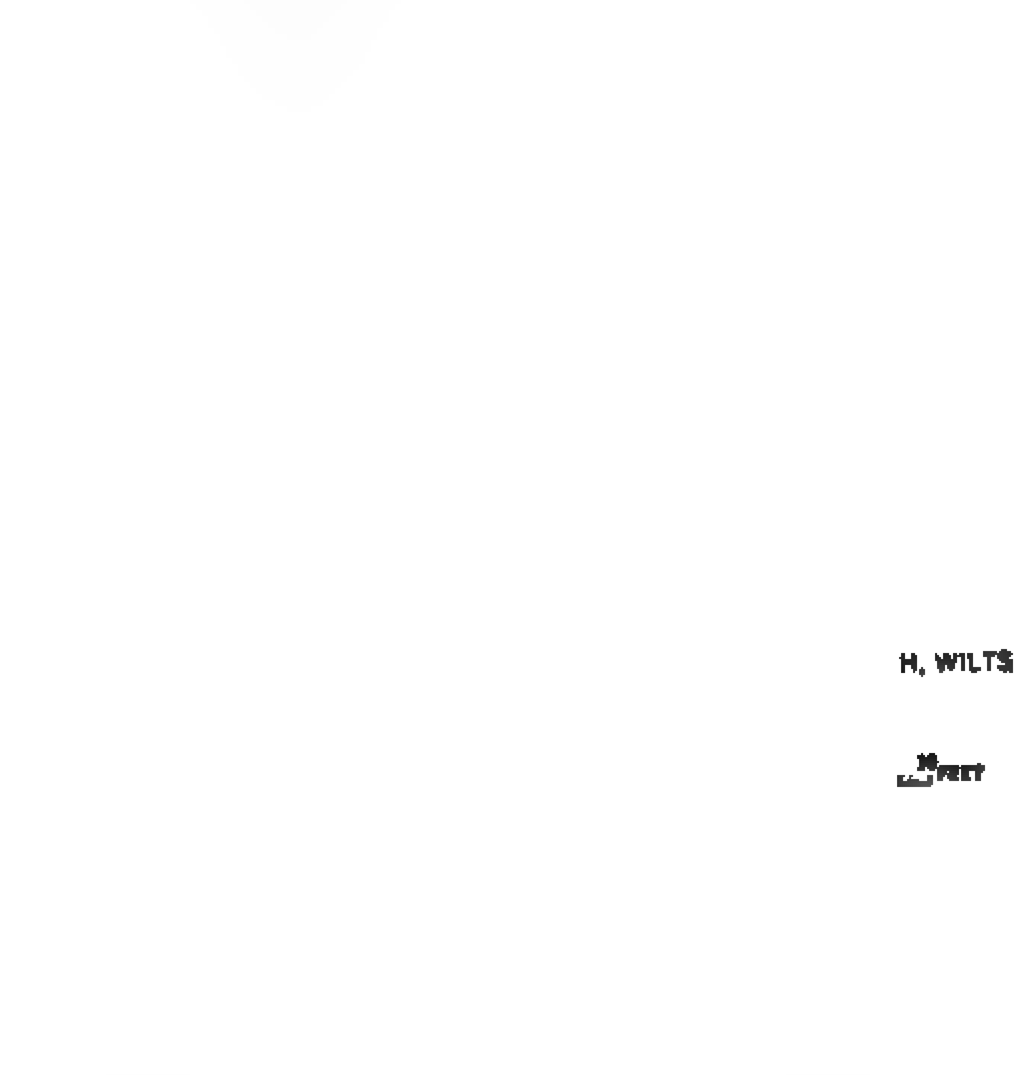
Fig. 30

Figs. 26–29 represent trusses taken from old English churches; but the hammer-beam truss is also frequently used in this country to support the roof of Gothic churches.

Fig. 30 represents half of one of the trusses in the First Church, Boston, Mass., Messrs. Ware & Van Brunt, architects. The truss is finished in black walnut, and has the effect of being very strong and heavy. Fig. 31 shows the framing of the same truss without any casing or falsework. It should be noticed that inside the

turned column, at the upper part of the truss (Fig. 30), there is an iron rod (Fig. 31) which holds up the joint A.[1]

In this form of truss the outward thrust of the arch enters the wall just above the corbel, K, and, as the direction of the thrust is inclined only about thirty degrees from a vertical, the tendency which it has to overthrow the wall is not very great, and may be easily resisted by a wall twenty inches or two feet thick, re-enforced by a buttress on the outside.



In trusses of this kind, the pieces should be securely fastened together whenever they cross or touch each other, and the whole truss made as rigid as possible. No dependence for extra strength should be made on the casings and panel-work.

Fig. 32 shows a truss derived from a hammer-beam truss, in which the ceiling is made to take the form of a vault. Trusses of

[1] The main rafters of this truss are two five-inch by thirteen-inch hard-pine timbers.

this kind, where there is no bracket under the hammer-beam, are not as stable as that shown in Fig. 30.

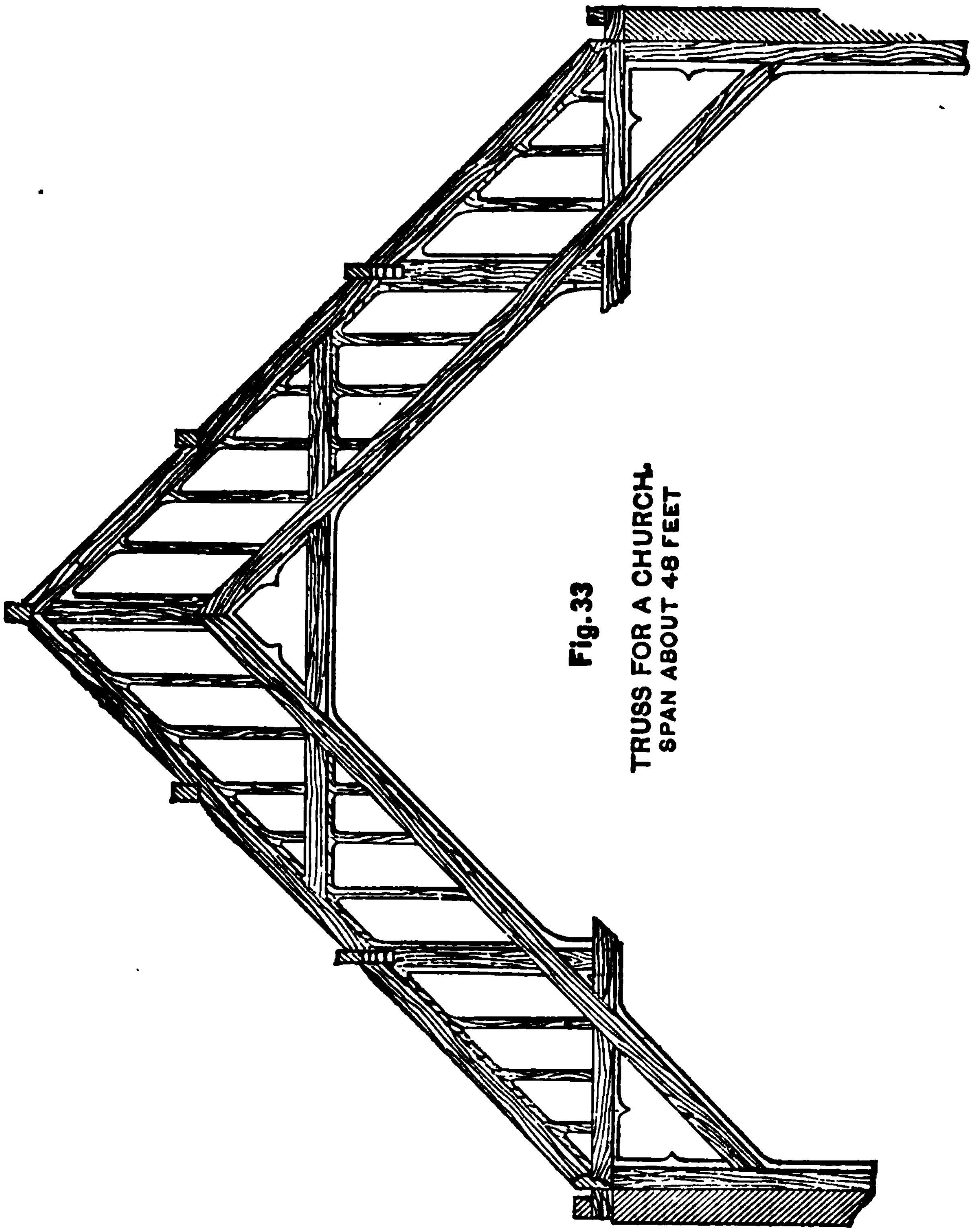
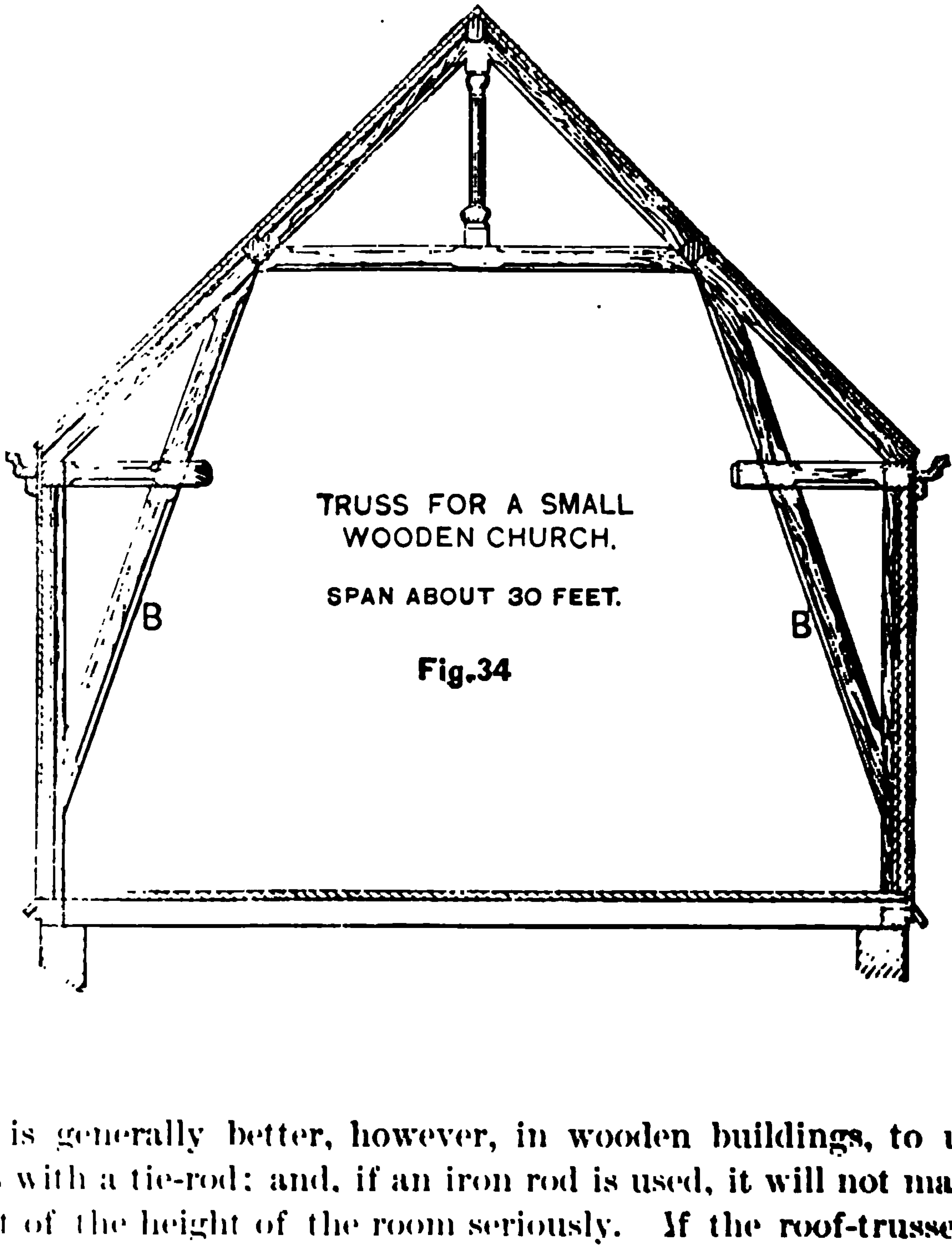


Fig. 33
TRUSS FOR A CHURCH.
SPAN ABOUT 48 FEET

Fig. 33 shows a form of truss used in Emmanuel Church at Shelburne Falls, Mass., Messrs. Van Brunt & Howe, architects, Boston. This truss was probably derived from the hammer-beam truss, and possesses an advantage over that truss in that it has in effect a trussed rafter, so that there is no danger of the rafter being broken; and, if the truss is securely bolted together at all its joints, it exerts but very little thrust on the walls. The rafters and cross-tie are formed of two pieces of timber bolted together, and the small upright pieces run in between them.

The trusses in the church at Shelburne Falls have the hammer-beams carved to represent angels.

Fig. 34 shows a form of hammer-beam truss sometimes used in wooden churches. The braces *BB* are carried down nearly to the floor, so that no outward thrust is exerted on the walls.



TRUSS FOR A SMALL WOODEN CHURCH.

SPAN ABOUT 30 FEET.

Fig. 34

It is generally better, however, in wooden buildings, to use a truss with a tie-rod: and, if an iron rod is used, it will not mar the effect of the height of the room seriously. If the roof-trusses are placed only about eight feet apart, the roof may be covered with two and a half inch spruce plank laid directly from one truss to the other without the intervention of jack-rafters or purlins. The planking can then be covered with slate or shingles on the outside, and sheathed within. Fig. 34 shows the roof covered in this way. Purlins are put in, however, flush with the rafters of the truss to divide the ceiling into panels.

Fig. 35 shows a section through the roof of St. James's Church, Great Yarmouth, Eng.

The span is thirty-three feet, and the trusses are spaced about eight feet apart from centres.

The size of the scantlings are as follows :—

Principals:	Rafters	12 inches	× 9 inches.
	Collars	9 "	× 9 "
	Ridge	12 "	× 5 "
	Purlins	8 "	× 5 "
	Cradling	7 "	× $2\frac{1}{2}$ "

The roof is constructed of Memel timber.

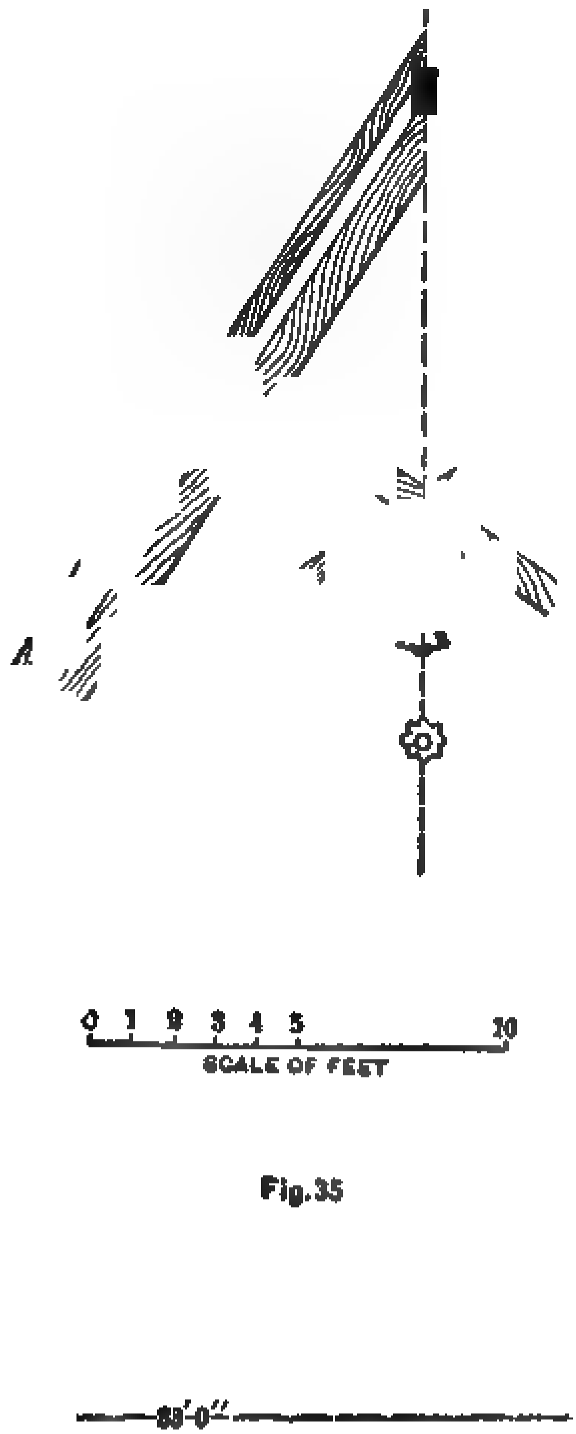


Fig. 35

CHAPTER XXVII.

IRON ROOFS AND ROOF-TRUSSES, WITH DETAILS OF CONSTRUCTION.

OWING to the increasing cost of lumber, and the necessity of erecting buildings as nearly fire-proof, and with as little inflammable material in the roof, as possible, it is becoming quite a common practice to roof large and expensive buildings with iron roofs, which, of course, involves the use of iron roof-trusses: hence it is important that the architect and progressive builder should have a general idea of the construction and principles involved in iron roof-trusses, and be familiar with the best forms of trusses for different spans, conditions of loading, etc.

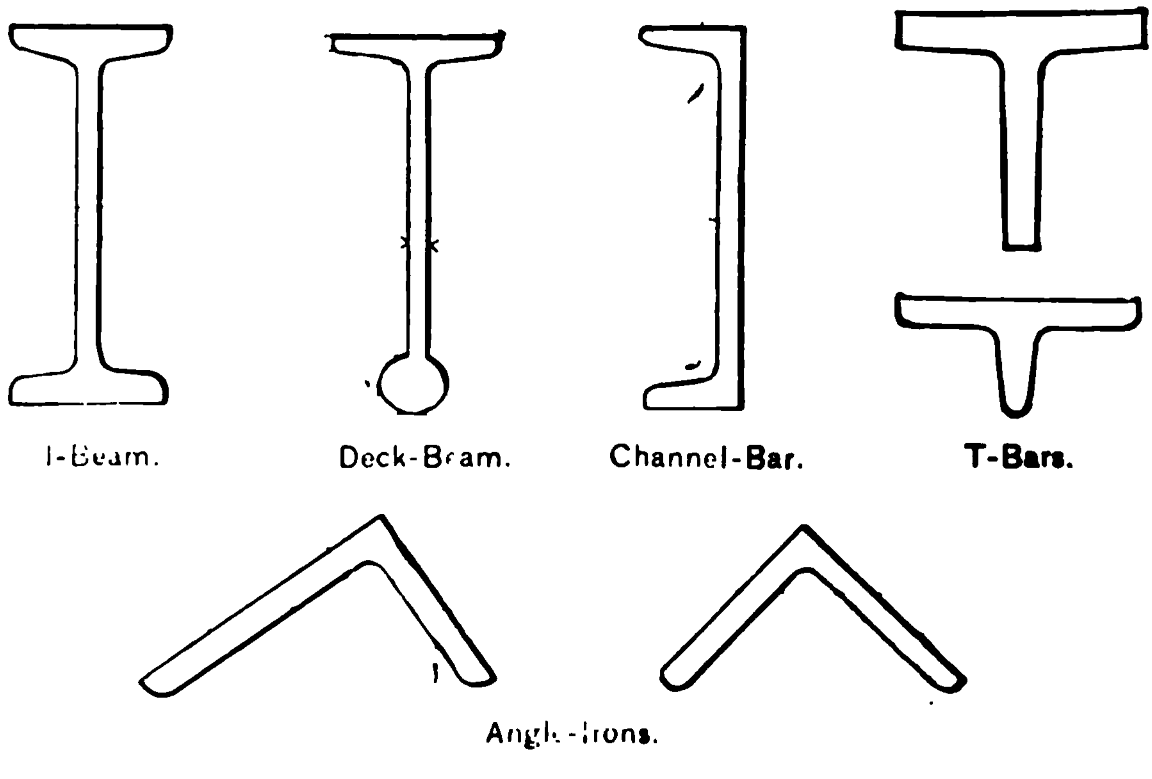


Fig. 1.

Besides being non-combustible, iron roof-trusses are superior to wooden trusses in that they may be built much stronger and lighter, and are much more durable.

Various forms of trusses have been constructed to suit different

conditions of span, load, height, etc., and of these the following examples have been found to be the best and most economical.

Before proceeding to describe these various forms of trusses, we would call the reader's attention to the sections of beams, angle-irons, T and channel bars, shown in Fig. 1. It will frequently be necessary to refer to these sections; as they are the principal shapes of rolled iron entering into the construction of iron roofs, and it is of great importance that an architect or builder be familiar with their forms and names.

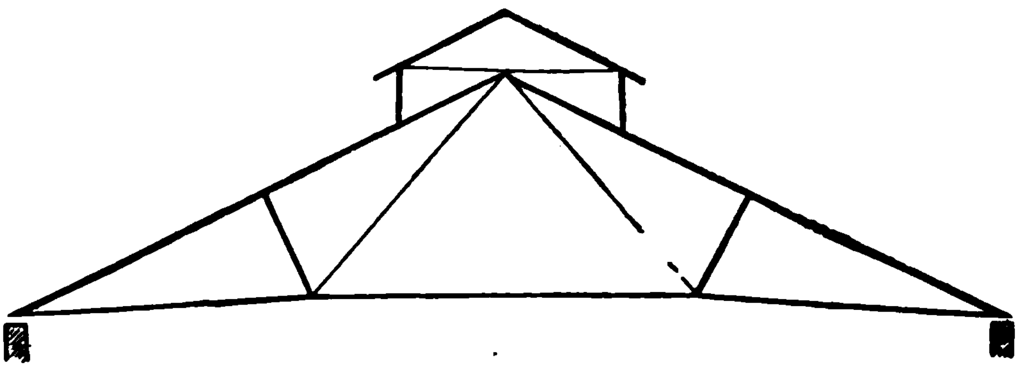
For convenience in describing the different forms of iron roofs, we shall divide them into the following classes:—

1st, *Truss-roofs with straight rafters*, which are simply braced frames or girders.

2d, *Bowstring-roofs with curved rafters* of small rigidity, and with a tie-rod and bracing.

3d, *Arched roofs*, in which the rigidity of the curved rafter is sufficient to resist the distorting influence of the load without additional bracing.

Trussed Roofs.—For small spans, the most economical and simplest form of truss is that represented in Fig. 2. (Owing to the



LEBANON FURNACE.

Fig. 2.

small scale to which it is necessary to draw these figures, we have represented the pieces by a single line, which has been drawn heavy for strut-pieces, and light for ties and rods.)

This truss was built by the Phœnix Iron Company for the roof of a furnace-building. It consists of two straight rafters of channel or T bars, two struts supporting the rafters at the centre, a main tie-rod, and two inclined ties assisting the tie-rod to support the end of the struts. The lines on the top of the truss represent the section of a monitor on the roof, which is not a part of the truss, but only supported by it.

One of the great merits of this truss is that it has but few pieces in compression, viz., the rafters and two struts; which is a condi-

tion very desirable in iron trusses, owing to the fact that wrought-iron resists a tensile strain much better than a compressive one, and hence it is more economical to use wrought-iron in the form of ties than in the form of struts.

It should be borne in mind that for *ties*, rods or flat bars of iron are the most suitable; while for *struts*, it is necessary to use some form of section that offers considerable resistance to bending, such as a T-iron, or four angle-irons riveted together in the form of a cross; for wrought-iron struts always fail by bending or buckling, and not by direct crushing. In Figs. 2–10 the pieces which are struts, or resist a compressive strain, are drawn with heavy lines, and those pieces which act as ties are drawn with a light line.

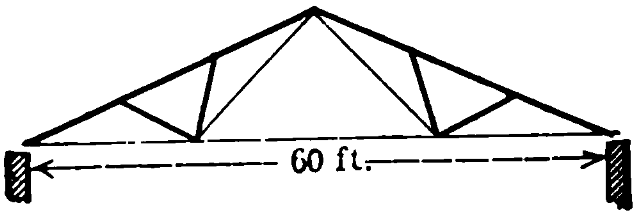
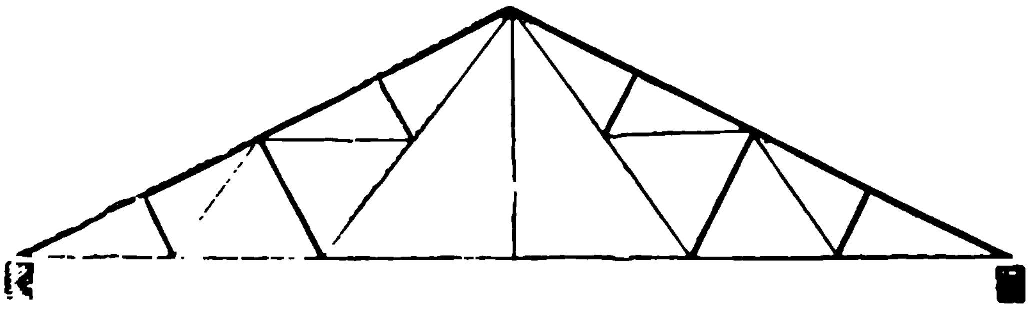


Fig. 3.

Fig. 3 represents a truss similar to that in Fig. 2, but having two struts instead of one, which is more economical where the span is over fifty-six feet, for the reason that it allows the rafters to be made of lighter iron.

For spans of from seventy to a hundred feet, the form of truss shown in Fig. 4 has been found to be about the most economical and satisfactory in every respect.

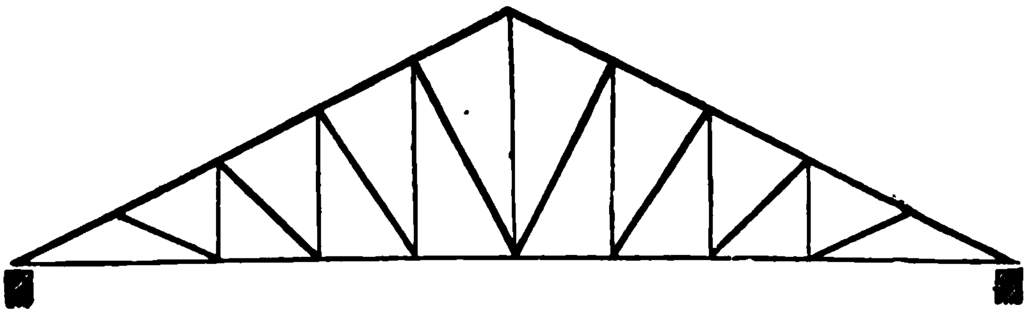


NEW MILL, PHŒNIX IRON-WORKS, ROCK-ISLAND ARSENAL.

Fig. 4.

The rafters in this truss, for moderate spans, may be T-irons; and for larger spans, channel-bars and the ties and struts may be bolted to the vertical rib. For very large spans, channel-bars may be used, placed back to back, with the ends of the bracing bars between them. I-beams are also used for the rafters, but they have the objection of not being in a shape to connect readily with the other forms of iron. The flanges of an I-beam do not offer so good an opportunity for riveting as do those of angle and T irons and

channel-bars. The ties are rods of round iron or flat bars; and the struts, commonly T-irons or angle-irons bolted together.



MASONIC TEMPLE, PHILADELHIA.

Fig. 5.

Another form of truss, shown in Fig. 5, derived from the wooden queen post truss, is very commonly used for spans of from sixty to a hundred and forty feet. A modification of this truss is shown in Fig. 6, in which both struts and ties are inclined, instead of only the

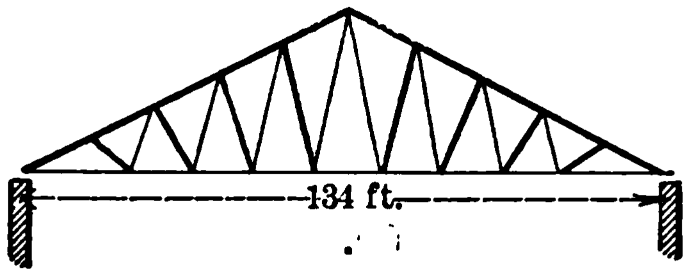
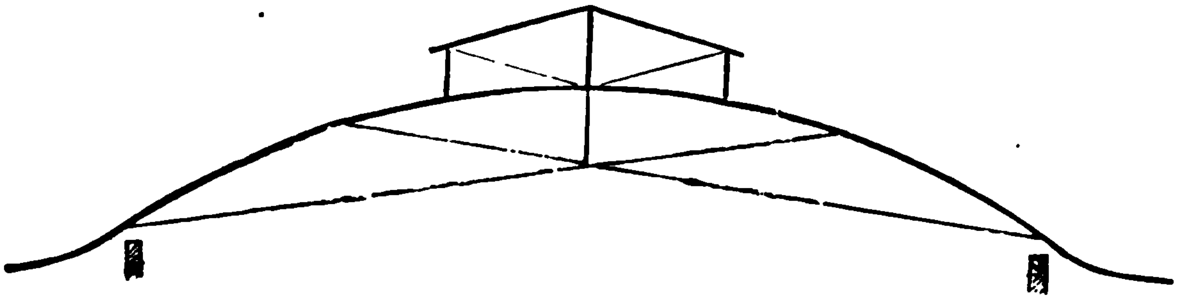


Fig. 6.

struts, as in Fig. 5. The truss in Fig. 6 has the advantage that the struts are shorter, more nearly perpendicular to the rafters, and less strained.

Bowstring-Roofs. — In designing iron roofs, it is sometimes desired to vary the ordinary straight pitch roof by using a curved rafter. Two examples of such roofs are shown in Figs. 7 and 8,



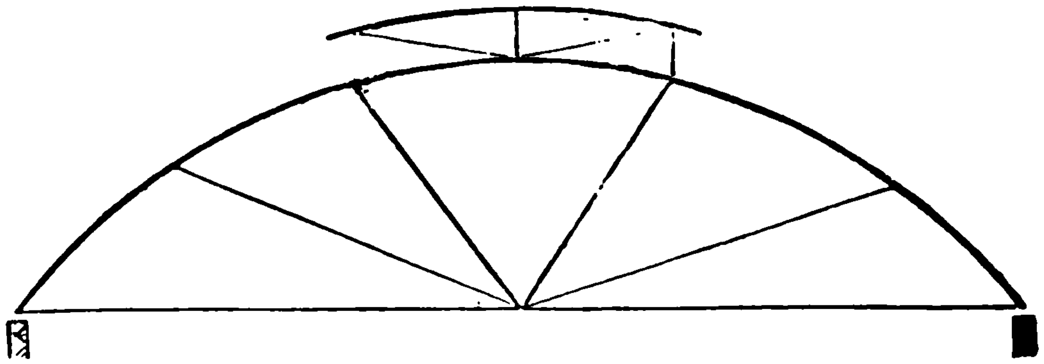
ALTOONA STATION, PENNSYLVANIA RAILROAD.

Fig. 7.

which were constructed by the Phœnix Iron Company of Philadelphia. These may be considered as the simplest forms of bowstring-roofs.

The principal use of the bowstring-roof proper is for roofing

very large areas in one span, such as is often desired in railway-stations, skating-rinks, riding-schools, drill-halls, etc.



MARKET-HOUSE, TWELFTH AND MARKET STREETS, PHILADELPHIA.

Fig. 8.

Fig. 9 represents the diagram of a bowstring-truss of a hundred and fifty-three feet span. The trusses in this particular case are spaced twenty-one feet six inches apart. The arched rafter consists of a wrought-iron deck-beam nine inches deep, with a plate, ten inches by an inch and a fourth, riveted to its upper flange. Towards the springing, this rib was strengthened by plates, seven inches by seven-eighths of an inch, riveted to the deck-beam on each side.

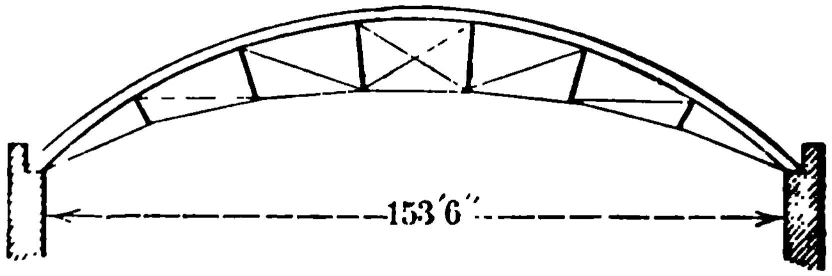


Fig. 9.

The struts are wrought-iron I-beams seven inches deep. The tie-rods have six and a half square inches area, and the diagonal tension-braces are an inch and a fourth diameter. These trusses are fixed at one end, and rest on rollers at the other, permitting free expansion and contraction of the iron under the varying heat of the sun.

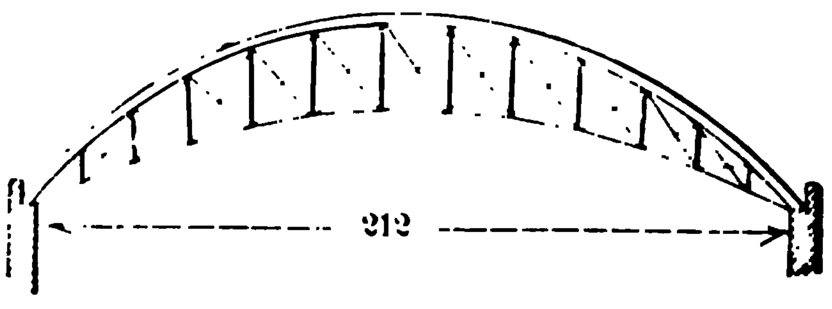


Fig. 10.

Fig. 10 shows a similar truss having a span of two hundred and twelve feet. It consists of bowstring principals spaced twenty-

four feet apart. The rise is one-fifth the span, the tie-rod rising seventeen feet in the middle above the springing, and the curved rafter rising forty feet and a half. The rafter is a fifteen-inch wrought-iron I-beam. The tie is a round rod in short lengths, four inches diameter, thickened at the joints. The tension-bars of the bracing are of plate-iron, five inches to three inches in width, and five-eighths of an inch thick. The struts are formed of bars having the form of a cross.

The following table, taken from Unwin's "Wrought-Iron Bridges and Roofs," gives the principal proportions of some notable bowstring-trusses, mostly in England: —

PROPORTIONS OF BOWSTRING-ROOFS.

For spans much exceeding a hundred and twenty or a hundred and thirty feet the bowstring-truss is much the most economical, and advantageous to use.

Arched Roofs. — These roofs consist of trusses in the form of an arch, having braced ribs, which possess sufficient rigidity in themselves to resist the load upon them. The thrust of these large ribs, however, has to be provided for, as in the case of masonry arches, either by heavy abutments or by tie-rods. As these trusses embrace the most difficult problems of engineering, and are rarely used, we have thought best not to give any examples of such trusses. If any reader should have occasion to visit the Boston and Providence Railroad Depot at Boston, he can there see an admirable example of this form of truss.

* At springing twenty-five square inches.

Details of Iron Trusses.

After deciding upon the form of truss which it will be best to use, the *shape* of the iron to form the different members is a matter to be considered. There are many practical reasons which make it desirable to use certain shapes of iron in constructing iron trusses, even though those shapes may not be the most desirable in regard to strength; so that a knowledge of the details of iron trusses is requisite for any one who wishes to become a master of building construction.

By far the best way to study the details of construction is to observe work already built and that which is in process of construction; but this requires considerable time, and often the thing one wants cannot be found at hand. The following details of the various ways of joining the different members of iron trusses will be found useful.

There are two general methods of constructing iron trusses. One is to make all the parts of the truss of combinations of angle-irons, channel-bars, and flat plates, and rivet them together at the joints, so that the truss will consist of a frame-work of iron bars all riveted together. The other method is to use channel-bars, T-irons, I-beams, etc., for the rafters and struts, and rods for the ties, which are connected at the joints by eyes and pins.

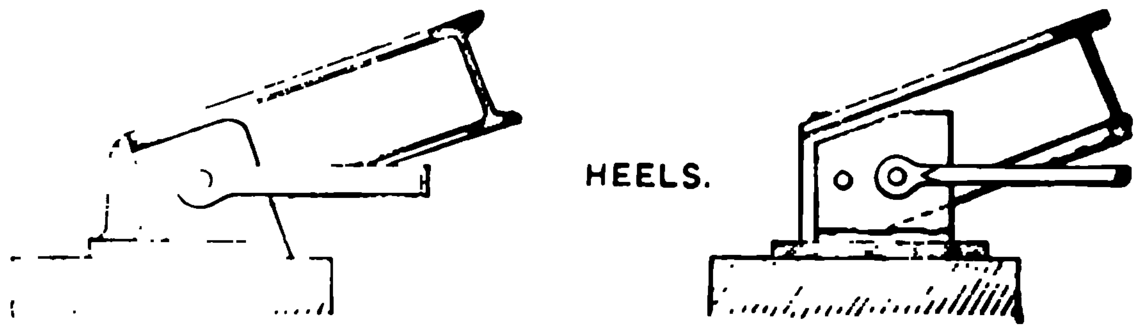


Fig. 11.

In the first method the ties are either made of flat bars or angle-irons.

Fig. 11 shows two ways in which the tie-rod is secured to the foot of the rafter in the second method of construction. A casting, forming a sort of "shoe," is made, in which the rafter fits, and the tie is secured to the "shoe" by means of an eye-end and pin; or a plate may be bolted to each side, and the whole rest on an iron plate. Of course the tie must in either case consist of two bars, one on each side of the shoe.

Fig. 12 illustrates two ways of fastening the upper ends of the struts to the rafters. In the first method the casting is made to fit inside the strut, and is bolted to the bottom of the rafter.

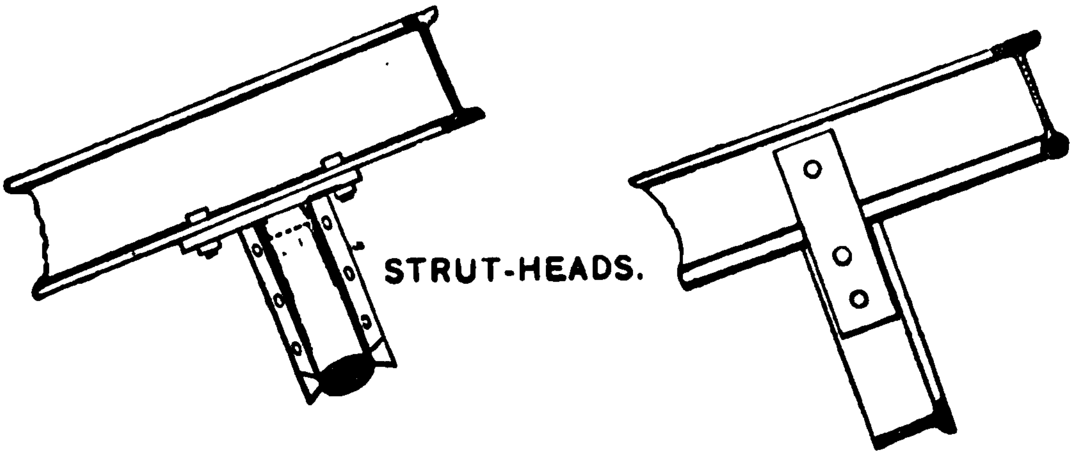


Fig. 12.

Fig. 13 shows the joints at the foot of the struts, as made in the

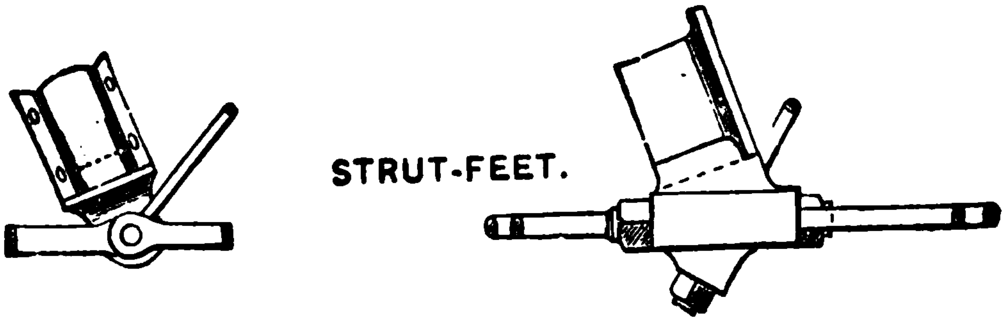


Fig. 13.

second method. The peaks in either method are secured by means of fish-plates riveted to both rafters (Fig. 14).

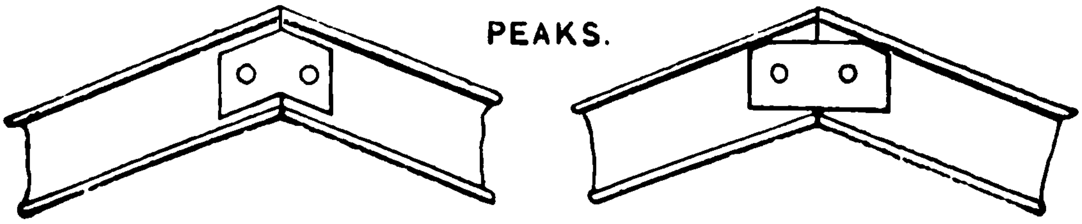


Fig. 14.

Fig. 15 shows the proportions for eyes and screw ends for tension-

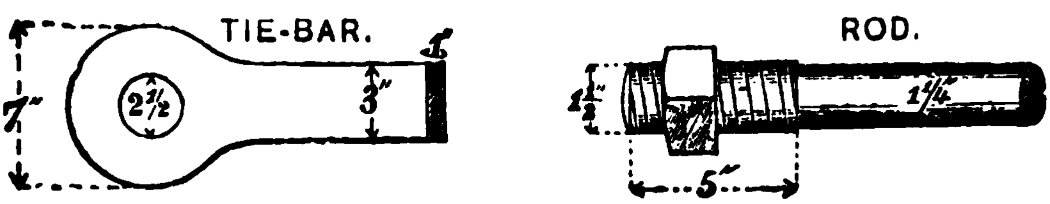


Fig. 15.

bars as used in this method of construction.

Figs. 16 and 17 show the manner of forming the joints in the first method of construction. Fig. 16 represents the joint at the

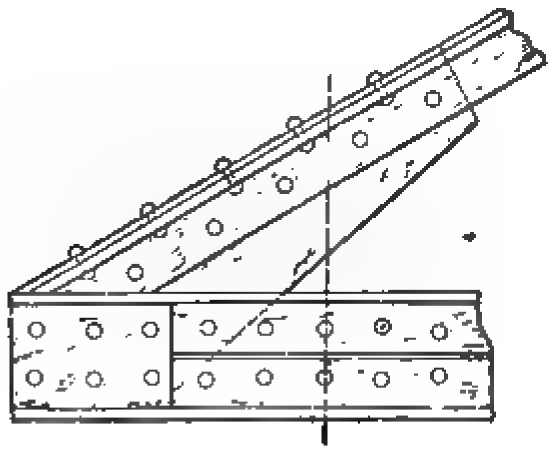


Fig. 16.

bottom of the main rafter; and Fig. 17, the joint where a rafter, straining-beam, tie, and strut come together. All the pieces are securely riveted to a piece of plate-iron, which thus holds them together. The other joints are formed in a similar way. Which is the *better* method of construction depends very much on circumstances.

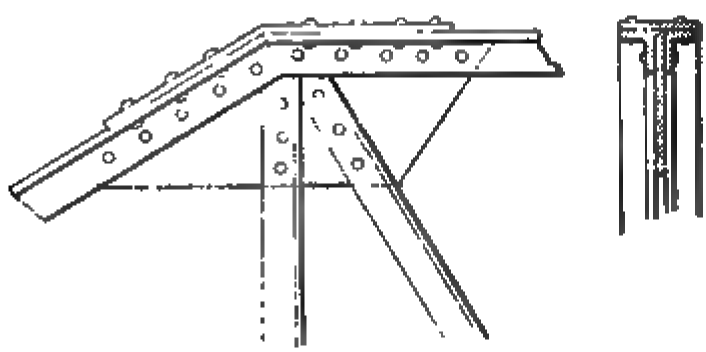


Fig. 17.

In roofs of wide span, provision for *expansion of the iron*, due to changes of *temperature*, may be made by resting the skewback of one end of the truss on a cast wall-plate, with rollers interposed to permit of the sliding of the shoe without straining the wall, as in Fig. 18; but this precaution is not necessary in roofs of sixty feet span or less. Careful experiments have proved that an iron rod one hundred feet long will vary about a tenth of a foot for a change of temperature of a hundred and fifty degrees F.; and, as this is the greatest range to which iron beams and rods in a building would probably be subjected in this climate, compensation to that amount would be sufficient for all purposes. For sixty feet span,

the vibration of each wall would then be only fifteen-thousandths of a foot either way from the perpendicular, — a variation so small, and so gradually attained, that there is no danger in imposing it upon the side-walls by firmly fastening to them each shoe of the rafter. Expansion is also provided against by fastening down one shoe with wall-bolts, and allowing the other to slide to and fro on the wall-plate without rollers.

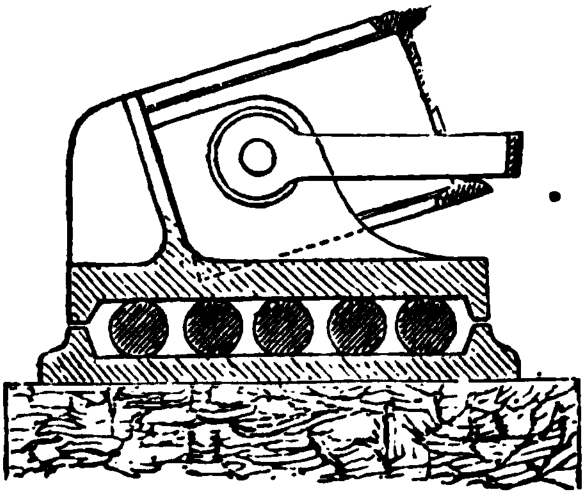


Fig. 13.

After the trusses are up, there are various ways of constructing the roof itself. If the roof is to be of slate, it is best to space the trusses about seven feet apart, and use light angle-irons for purlins, which are spaced from seven to fourteen inches apart, according to the size of the slate. On the iron purlins the slate may be laid directly, and held down by copper or lead nails clinched around the

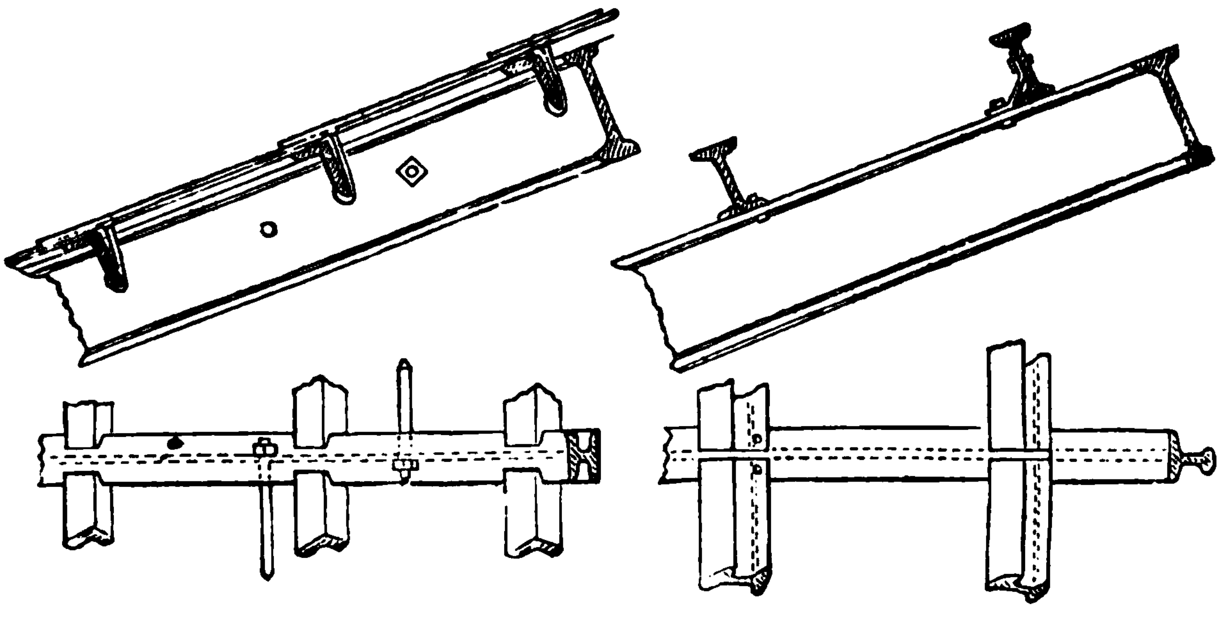


Fig. 19.

angle-bar; or a netting of wire may be fastened to the purlins, and a layer of mortar spread on this, in which the slates are bedded. When greater intervals are used in spacing rafters, the purlins may be light beams fastened on top or against the sides of the principals

with brackets, allowance always being made for longitudinal expansion of the iron by changes of temperature. On these purlins are fastened wooden jack-rafters, carrying the sheathing-boards or laths, on which the metallic or slate covering is laid in the usual manner; or sheets of corrugated iron may be fastened from purlin to purlin, and the whole roof be entirely composed of iron.

When the rafters are spaced at such intervals as to cause too much deflexion in the purlins, they may be supported by a light beam placed midway between the rafters, and trussed transversely with posts and rods. These rods pass through the rafters, and have bevelled washers, screws, and nuts at each end for adjustment. By alternating the trusses on each side of the rafter, and slightly increasing the length of the purlins above them, leaving all others with a little play in the notches, sufficient provision will be made for any alteration of length in the roof, due to changes of temperature.

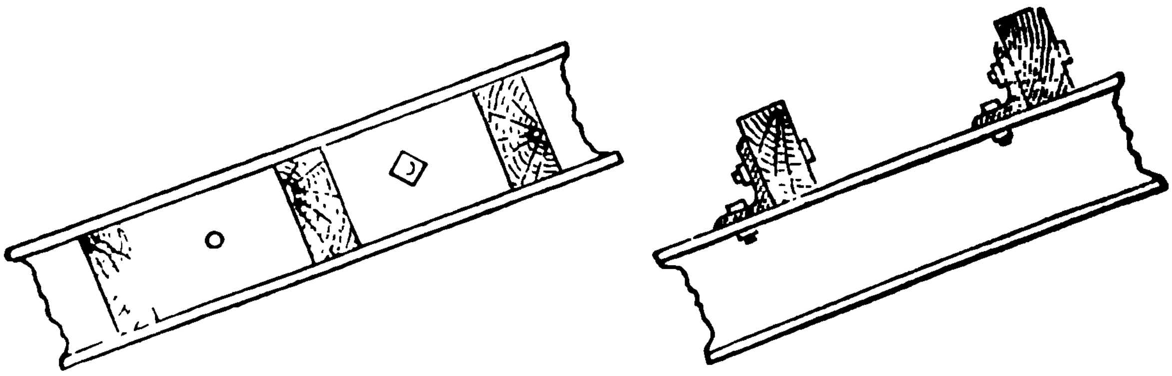


Fig. 20.

When wooden purlins are employed, they may be put between the rafters, and held in place by tie-rods on top, and fastened to the rafters by brackets; or hook-head spikes may be driven up into the purlin, the head of the spike hooking under the flange of the beam, spacing-pieces of wood being laid on the top of the beam from purlin to purlin. The sheathing-boards and covering are then nailed down on top of all in the usual manner.

CHAPTER XXVIII.

THEORY OF ROOF-TRUSSES.

In this chapter it is proposed to give practical methods for computing the weight of the roof with its load, and the proportion of the truss and its various parts.

The first step in all calculations for roofs is to find the exact load which will come upon each truss, and the load at the different joints. The load carried by one truss will be equal to the weight of a section of the roof of a width equal to the distance between the trusses, together with the weight of the greatest load of snow that is ever likely to come upon the roof. In warm climates, of course, the weight of snow need not be provided for.

It is a very common practice to assume the maximum weight of the roof and its load at from forty to sixty pounds per square foot of surface; but, while this may be sufficiently accurate for wooden roofs, it would hardly answer for iron roofs, where the cost of the iron makes it desirable to use as little material in the truss as will enable it to carry the roof with safety, and no more. The weight of the roof itself can be easily computed, and a sufficiently accurate allowance can be made for the weight of the truss; and, if the roof is to be in a climate where snow falls, a proper allowance must be made for that; and, lastly, the effect of the wind on the roof must also be taken into account.

Mr. Trautwine says, that within ordinary limits, *for spans not exceeding about seventy-five feet*, and with trusses seven feet apart, the total load per square foot, including the truss itself, purlins, etc., complete, may be safely taken as follows:—

Roof covered with corrugated iron, unboarded . . .	8	pounds.
If plastered below the rafters	18	"
Roof covered with corrugated iron or boards	11	"
If plastered below the rafters	18	"
Roof covered with slate, unboarded, as on laths . . .	13	"
Roof covered with slate on boards $1\frac{1}{4}$ inches thick . .	16	"
Roof covered with slate, if plastered below the rafters .	26	"
Roof covered with shingles on laths	10	"
If plastered below the rafters, or below tie-beam .	20	"
Roof covered with shingles on $\frac{7}{8}$-inch board	13	"

wind: hence the resultant of the wind pressure must act in a direction normal (at right angles) to the face of the roof. In this country the wind seldom blows with a pressure of more than forty pounds per square foot on a surface at right angles to the direction of the wind ; and it is considered safe to use that number as the greatest wind pressure.[1] But the pressure on the roof is generally much less than this, owing to the inclination of the roof. The following table gives the normal wind pressure per square foot on surfaces inclined at different angles to the horizon, for a horizontal wind pressure of forty pounds per square foot.

NORMAL WIND PRESSURE.

Angle of Roof.		Normal pressure.	Angle of Roof.		Normal pressure.
Degrees.	Rise in one foot.		Degrees.	Rise in one foot.	
5	1 inch.	5.2 lbs.	35	$8\frac{2}{5}$ inches.	30.1 lbs.
10	$2\frac{1}{8}$ inches.	9.6 "	40	10 "	33.4 "
15	$3\frac{1}{5}$ "	14.0 "	45	12 "	36.1 "
20	$4\frac{3}{8}$ "	18.3 "	50	$14\frac{3}{10}$ "	38.1 "
25	$5\frac{1}{2}$ "	22.5 "	55	$17\frac{1}{8}$ "	39.6 "
30	$6\frac{9}{10}$ "	26.5 "	60	$20\frac{3}{4}$ "	40.0 "

Until of late years it has been the general custom to add the wind pressure in with the weight of snow and roof ; and, although this is evidently not the proper way to do, yet for wooden trusses it gives results which are perhaps sufficiently accurate for all practical purposes ; and, if caution is taken to put in extra bracing wherever any four-sided figure occurs, this method will answer very well for wooden trusses. For iron trusses, however, the strains in the truss due to the vertical load on the truss, and those due to the wind pressure, should be computed separately, and then combined, to give the maximum strains in the various pieces of the truss. It should be borne in mind that a horizontal wind pressure of forty pounds per square foot is quite an uncommon occurrence, and, when it does occur, generally is of short duration ; so that a truss which would not withstand this pressure, if applied for a long

[1] At the observatory, Bidston, Liverpool, the following wind pressures per square foot have been registered. 1868, Feb. 1, 70 pounds; Feb. 22, 65 pounds; Dec. 27, 80 pounds. 1870, Sept. 10, 65 pounds; Oct. 13, 65 pounds. 1871, March 9, 90 pounds. 1875, Sept. 27, 70 pounds. 1877, Jan. 30, 63 pounds; Nov. 23, 63.5 pounds.—American Architect, vol. xv. p. 237.

time, may possess sufficient elasticity to withstand the strain for short time without injury.

In very exposed positions, such as on high hills or mountain: where the force of the wind is unobstructed, the roofs of all hig buildings should be especially designed to withstand its powerfu effects.

Graphical Analysis of Roof-Trusses.—The simplest and in most cases the readiest, way of computing the strains i trusses, is by the graphic method, which consists in representin the loads and strains by lines drawn to a given scale of pound to the fraction of an inch.

We think the graphic analysis of roof-trusses may be best show by examples, and hence shall give a sufficient variety to show th method of procedure for most of the trusses already described i these articles.

EXAMPLE 1.—As the simplest case, we will take the trus shown in Fig. 4, Chap. XXVI.

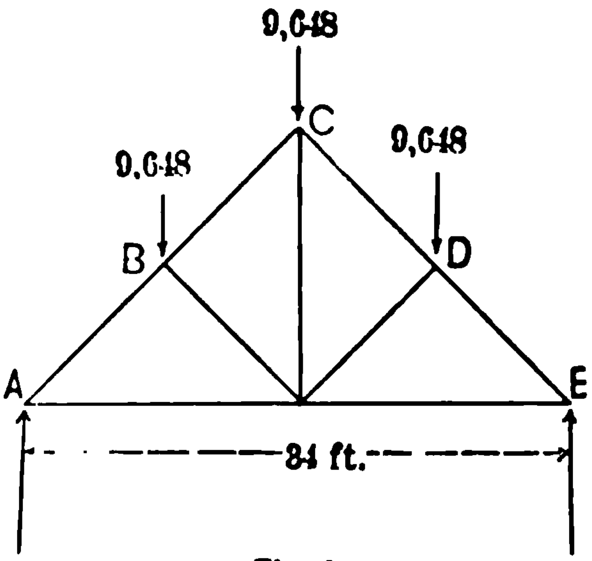


Fig. 1.

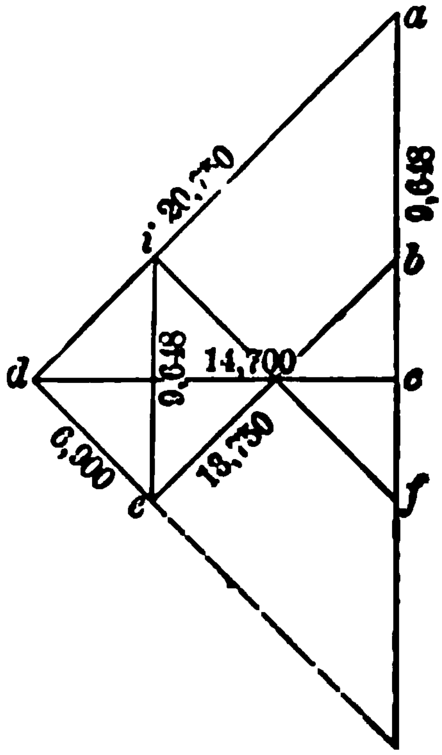


Fig. 1a.

If we should draw a line through the centre of each piece of thi truss, we should have a diagram such as is shown in Fig. 1. W will suppose that this truss has a span of 34 feet, and the rafter have an inclination of 45° with a horizontal line. Then the lengt of the rafter would be 24 feet; and, if the trusses were 12 feet apart one truss would support a roof-area of 12 × 24 × 2 = 576 squar feet. Now, if we look at Fig. 1, we can see that the purlin or plat at *A* or *E* would carry one-half of the roof from *A* to *B*. The pu lin at *B* would carry the roof from a point midway between *A* an *B* to a point midway between *B* and *C*, which would be one-fourt the area of roof supported by each truss.

The purlins C and D would also support the same amount of roof.

If we consider the roof to be slated on boards an inch and a fourth thick, we shall have for the weight of one square foot 16 pounds ; allowing for snow, 15 pounds ; normal pressure of wind, 36 ; total weight or load on one square foot, 67 pounds ; total weight supported by one truss, $67 \times 576 = 38{,}592$ pounds ; total load coming at each of the points B, C, and D, one-fourth of $38{,}592 = 9648$ pounds.

The load coming at A and E is supported directly by the walls of the building, and need not be considered as coming on the truss at all. If, now, we draw a vertical line on our paper, and, commencing at the upper end, lay off 9648 pounds at some convenient scale, say 5000 pounds to the inch (in the following figures different scales have been used to keep the diagrams within the limits of the page, but were first drawn to a large scale to get thes tresses more accurately), and then one-half of 9648 pounds, or 4824 pounds, to the same scale, we shall have the line ac (Fig. 1a) representing just half the load on the truss, or the load coming on each of the supports.

Now, that the forces acting in the rafter and tie-beam, and the supporting forces, all coming together at the point A, shall balance each other, they must be in such a proportion, that if we draw a line from a parallel to the rafter, and a line through c parallel to the tie-beam, the line ad must represent the thrust in the lower part of the rafter, and the line dc, the pull in the tie-beam. If we next consider the forces acting on the joint B, commencing with the rafter, and going around to the right, we find that the first force which we know, is the force in the rafter, represented in Fig. 1a by the line da. Next we have the weight, 9648 pounds, acting down, represented by the line ab, and there remain two unknown forces,—that in the upper part of the rafter and the force in the strut.

To obtain these forces, draw a line through b (Fig. 1a), parallel to the rafter, and a line through d, parallel to the strut. These two lines will intersect in e; and the line be will represent the force in the rafter, and the line ed the force in the strut. Furthermore, if we follow the direction in which the forces act, we shall see that the force da acts up : hence the rafter is in compression. The remaining forces must act around in order : hence ab acts down, be acts towards the joint, and ed acts up towards the joint, so that both pieces are in compression.

Next take the forces acting at the point C. The first force we know is eb, which acts up ; next we have the weight, 9648 pounds,

which would extend beyond e to f; then there remain the forces in the rafter to the right, and the vertical tie, which are determined by drawing a line through f parallel to the rafter, and a line through e parallel to the tie. These two lines intersect in i; and the line if will represent the force in the rafter, and ei will represent the pull in the tie. We have now only to measure the lines in our diagram of forces, and we have the forces acting in every part of the truss; as, of course, the corresponding pieces on the different sides of the truss would be similarly strained. Measuring the different force-lines by the same scale we used in laying off the weight, we find the strains as shown by the figures on the lines, Fig. 1*a*.

Having found the strain-pressure in the different parts of the truss, it is very easy to determine what should be their dimensions.

Thus the compression in the foot of the rafter is 20,750 pounds. Now, if we wish to make it of hard pine, we know that hard pine will safely bear 1000 pounds to the square inch; and hence we shall need $\frac{20750}{1000} = 21$ square inches area in the rafter. This would require only a 3 by 7 timber; but, as the rafter will need to be cut into more or less, we will give it more area, and call it a 6 by 6.

The short struts have a pressure of 6,000 pounds, and hence need not be larger than a 2 by 4, except, that, being so thin, it is liable to bend; and so we will make it 4 inches by 6 inches. The tie-beam resists a pull of 14,700 pounds; and, as hard pine will safely withstand a tensile strain of 2000 pounds, we should only need about eight square inches of area; but, while this would resist the pull, we must add enough more to allow for cutting into the tie at the joints, and for sagging under its own weight; so that we will make the beam out of a 6 inch by 6 inch timber.

The centre tie, which has to resist a pull of 9648 pounds, we will make of wrought-iron instead of wood, as shown in Fig. 4, Chap. XXVI.; and, as wrought-iron may be safely trusted with a pull of 10,000 pounds to the square inch of cross-section, we shall need a rod having a sectional area of not quite one square inch, or a rod of an inch and an eighth, or an inch and a fourth, in diameter.

If the rafter and strut had been of spruce, we should have divided the strain by 800 pounds, or 700 if of white pine; and for the tie we should have divided the pull by 1800 if spruce was to be used, and by 1500 if we intended to use white pine.

It will be noticed, that, while we determine the size of our timbers mathematically, it often happens that we must make them considerably larger to prevent their bending under their own weight, and to allow for cutting, boring, splicing, etc.; so that it will not do to depend entirely upon mathematical deductions, but

these should be supplemented by a practical knowledge of the subject.

The methods of determining the strains in this truss applies to all trusses properly put together, and which do not exert an outward thrust on their supports.

EXAMPLE 2.—For further illustration we will take the truss shown in Fig. 5, Chap. XXVI., and of which a diagram is given

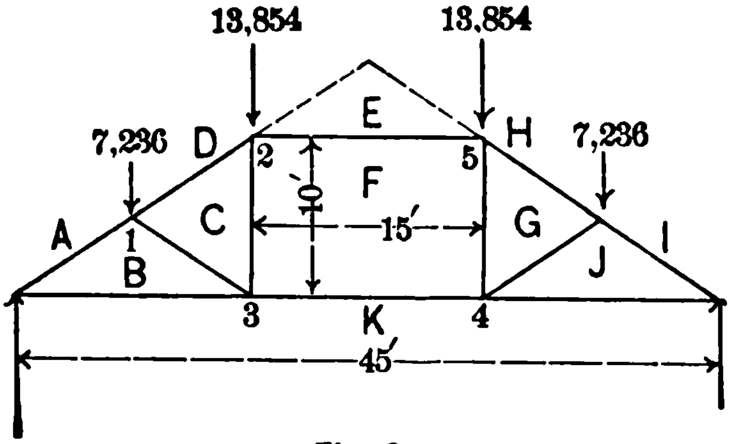


Fig. 2.

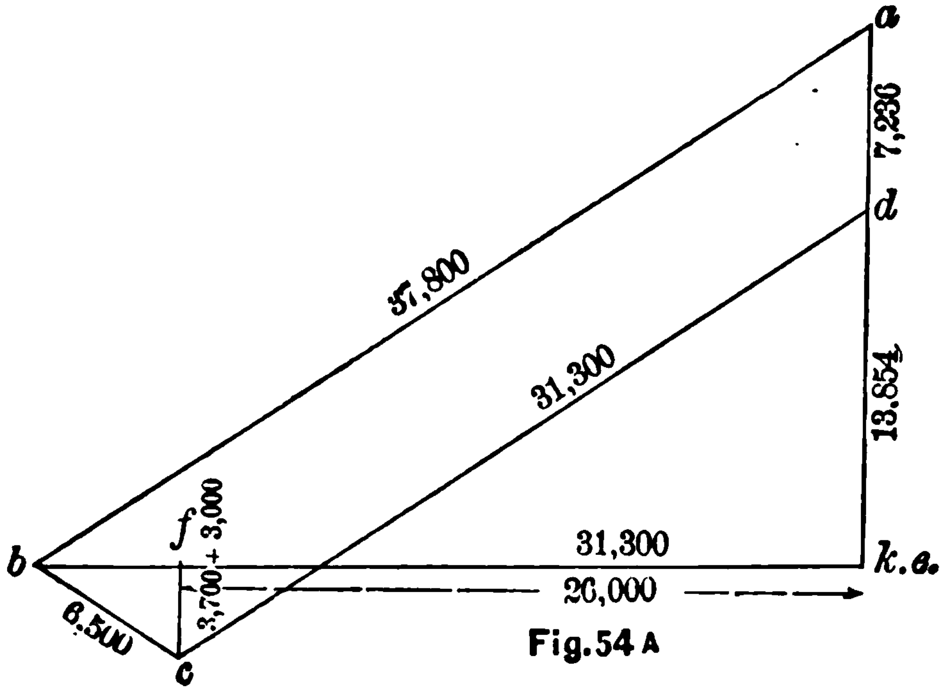


Fig. 2a.

in Fig. 2. We will assume that it has a span of 45 feet, and other dimensions as given in the figure; also that the trusses are placed 12 feet apart from centres. By glancing at Fig. 5, Chap. XXVI., it will be seen that the purlin at 2 (Fig. 2) carries the weight of that portion of the roof extending from halfway between purlins 1 and 2 to the ridge of the roof, and in this case equal to $13\frac{1}{2} \times 12 = 162$ square feet. The purlin at 1 supports the roof for $4\frac{1}{2}$ feet each side of it, or $9 \times 12 = 108$ square feet. This would bring a pressure of 10,854 pounds at the joint 2, and 7236 pounds at joint 1. Besides this, we have a ceiling suspended from the tie-beams of the truss, which would weigh about twenty pounds to the

square foot more. This weight would be supported one-third at each of the joints 3 and 4, and one-sixth at each end of the truss. The weight of the ceiling, coming at joints 3 and 4, may be assumed to be hung from joints 2 and 5 by means of the vertical rods: so we can add the weight coming from the ceiling to the weight of the roof, and consider it as applied at the points 2 and 5. The whole area of the ceiling is $12 \times 45 = 540$ square feet, and its weight about 9000 pounds; making 3000 pounds applied at 3 and 4, and the total load at 2 and 5, 13,854 pounds. The load at 1 we have already determined to be 7236 pounds. This gives us sufficient data with which to draw our diagram of strains.

As in Example 1, first lay off the loads on a vertical line, to some convenient scale: thus, *ad* (Fig. 2*a*), load at first purlin, 1, and *de*, the loads at 2 and 3 combined. Then *ae* represents half the weight supported by the truss, and also the load coming upon each support.

To draw the strains, first draw *ab* (Fig. 2*a*) parallel to *AB* (Fig. 2), and a horizontal line through *e*, intersecting *ab* in *b*; next go to the joint 1, and we have the force *ba*, acting upwards; then the load *ad*; then from *d*, the stress in *DC*, which must act in a direction parallel to it, and the stress in *BC*, also acting parallel to it. These last two stresses are found by drawing a line through *d* parallel to *DC*, and a line through *b* parallel to *BC*.

NOTE. — In Fig. 2 the lines are denoted by the letters either side of them; thus the bottom of the rafter on the left is called *AB*, and the brace *BC*; the [illegible] is denoted *CF*, and the right one *FG*. In the diagram of strains, the line representing the strain in any piece is denoted by *the same letters as the piece*, with the difference that small letters are used for the strain diagram, and the letters come at the ends of the lines. This method of notation (known as "Bow's Notation"), is very convenient, and aids greatly in following out the strains.

Next take the strains in the pieces at the joint 3. We know already the strains *kb* and *be*, and drawing the line *ef* parallel to *EF*, and *kf* parallel to *KF*, we have the strains in the remaining pieces. It will be noticed that the line *ef* lies over the line *eb*; but it should be kept in mind that they represent two separate strains, and should be measured separately.

Considering next the strains at joint 2, we find we already have [illegible], and *de* (13,254 pounds), leaving only *kf* to close the [illegible], showing that the strain in the beam *EF* is the same as that in the tie *Fk*, though the former is a compressive strain, and the latter a pulling one.

We now have the strains in all the pieces of the truss, represented by the corresponding lines in Fig. 2*a*, and, measuring these

by our scale of pounds, we find them to be as shown by the figures on the lines in the strain diagrams.

Then, if the truss were to be built of spruce, we should need $\frac{37800}{800} = 47$ square inches of section, at least in the main rafter, $\frac{26800}{800} = 33$ square inches, in the straining-beam, and $\frac{31300}{1800} = 18$ square inches, in the end of the tie-beam. Knowing these least dimensions, we can modify them to allow for cutting, joints, sagging, etc., according to our judgment. Thus we would make the rafters and straining-beam 6 inches by 10 inches, the tie-beam 6 inches by 10 inches, and the braces 6 inches by 6 inches. The rods have a strain of 3700 pounds plus the direct pull of 3000 pounds; making 6700 pounds' pull on the rod, which would require a rod one inch in diameter.

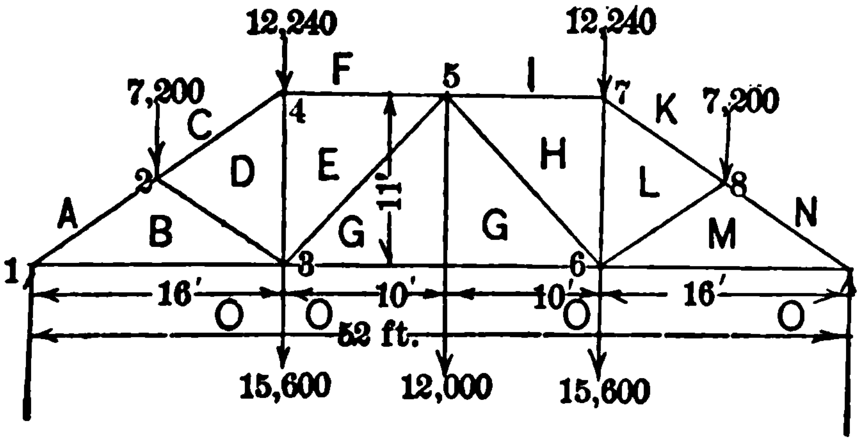


Fig. 3.

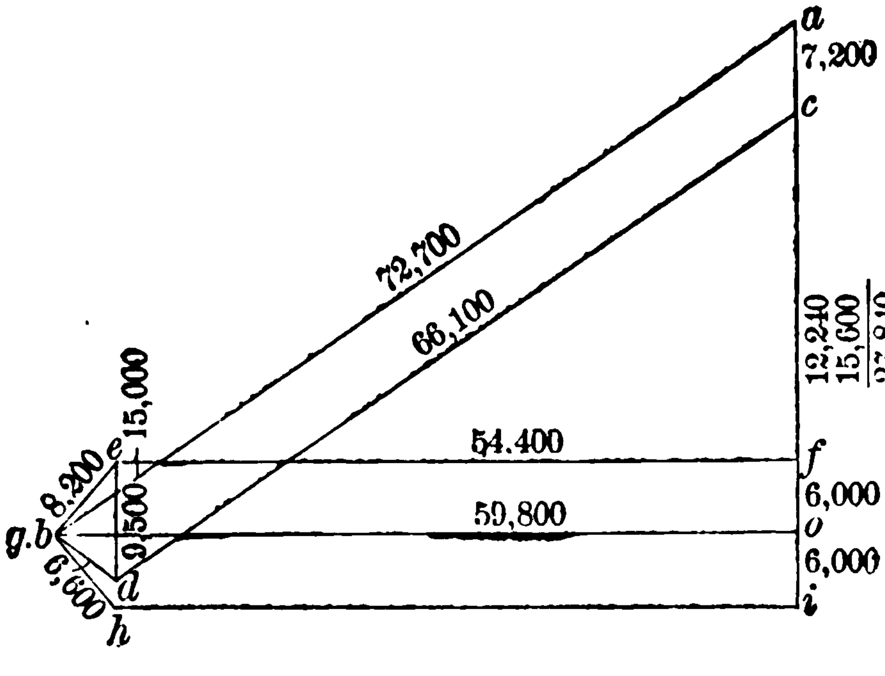


Fig. 3a.

EXAMPLE 3. — Take the truss represented by the diagram shown in Fig. 3, loaded with the weight of the roof, and supporting the floor below by means of rods suspended from joints 3, 5, and 6. The loads at the various joints would be about as given in the figure.

joint 7 we have *gm* and *mn* (18,320 pounds), and draw *nh* and *gh* to close the figure. This completes the strains in all the pieces

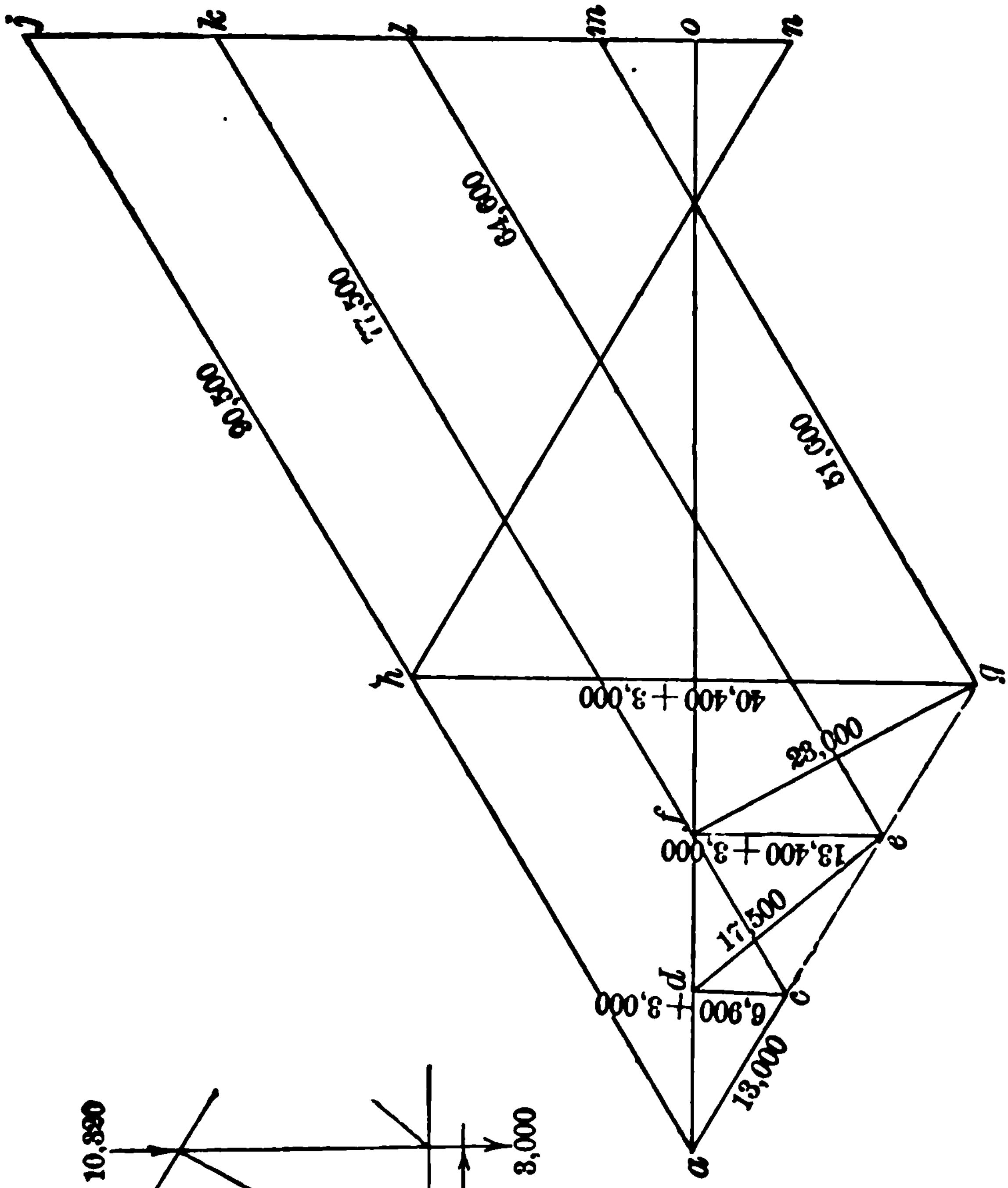


Fig. 4a.

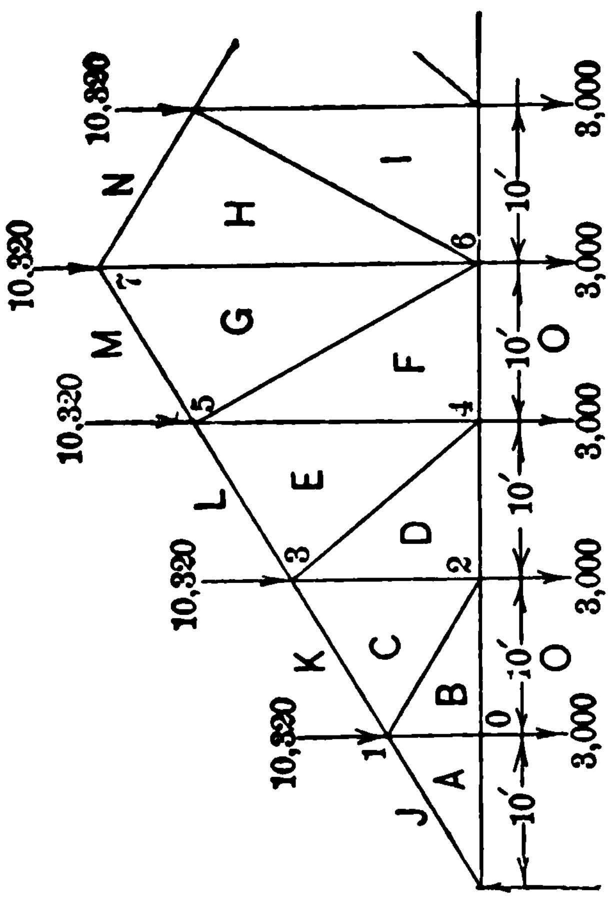
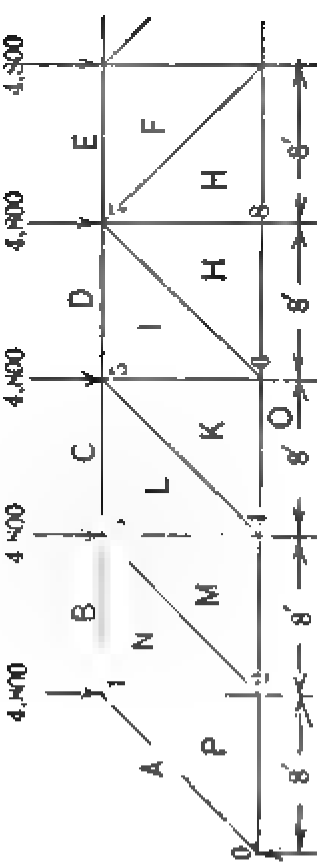


Fig. 4.

for one-half of the truss, and of course the strains for each half are the same.

It is obvious, that, as far as finding the strains is concerned, it makes no difference whether the truss be of iron or wood; the difference in the material only being taken into account when the sizes of the various pieces are determined.

EXAMPLE 5 (*Truss with Horizontal Chords*). — For the next example, we will take a truss like that shown in Fig. 15, Chap. XXVI., and of which a skeleton is shown in Fig. 5. This truss is



for a span of sixty-four feet, and supports a flat roof and plaster ceiling below the tie-beam, and also a gallery below on each side. The loads at the different joints would be about as indicated in Fig. 5. To draw the strain diagram (Fig. 5*a*), lay off the loads on a vertical line, commencing first with the loads nearest the support. Thus, *ab* equals load at joints 1 and 2, *bc* equals load at joints 3 and 4, *cd* equals load at joints 5 and 6, and *do* and *oe* each equals one-

half of loads at 7 and 8, because one-half of this load is borne by each support.

Next, commencing at joint *o*, we have the supporting force *oa*, the stress in the rafter *ap*, and the stress in the tie, *po*, closing the figure. At joint 1 we know *pa* and *ab*, and draw *bn* and *pn*, closing the figure. At joint 2 we know *op* and *pn* already, and draw *nm* and *om*. At joint 3 we know *mn*, *nb*, and *bc*, and draw *cl* and *ml*. The strains at joints 5 and 6 are found in the same way as those at 3 and 4; and at joint 7 we know the strains *hi*, *id*, and *de*, and draw *ef* and *hf*. The centre rod *HH* has no strain excepting the direct pull of 2400 pounds, so it cannot be represented in the diagram of strains.

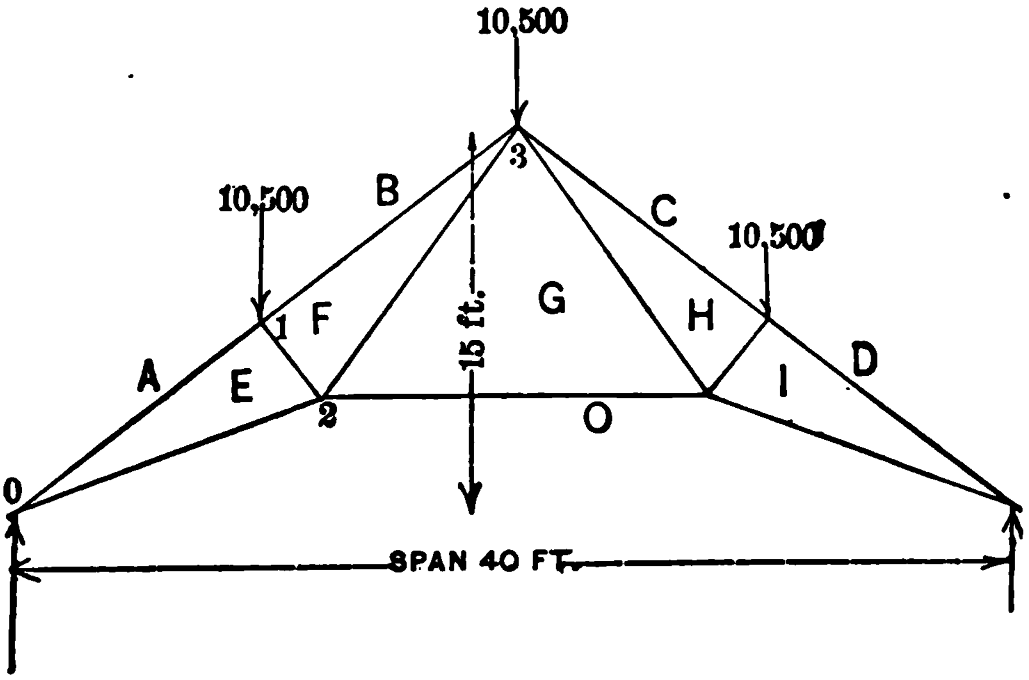


Fig. 6.

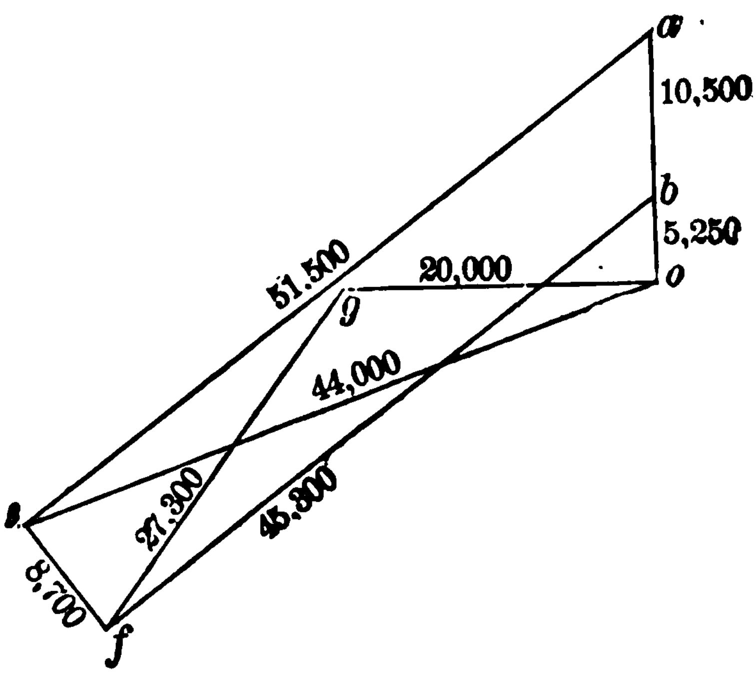


Fig. 6a.

The strains, in pounds, in the various pieces, are given in figures on the strain diagram.

EXAMPLE 6. — Truss such as shown in Fig. 17, Chap. XXVI., being a combination of iron and wood truss, suitable for a large shed or stable. The skeleton of this truss is shown in Fig. 6; the span in this example being forty feet, and the rise fifteen feet. The loads from the weight of the roof would be about as indicated in Fig. 6, there generally being no ceiling in roofs of this kind.

The diagram of strains is shown in Fig. 6*a*. *ab* equals load at joint 1, and *bo* equals one-half load at joint 3; *oa*, *ae*, and *oe* represent the strains at joint *o*; *ea*, *ab*, *bf*, and *fe*, the strains at joint 1; and *oe*, *ef*, *fg*, and *go* represent the strains at joint 2, completing all the strains in the truss. The complete diagram of strains for both sides of the truss is shown in Fig. 6*b*.

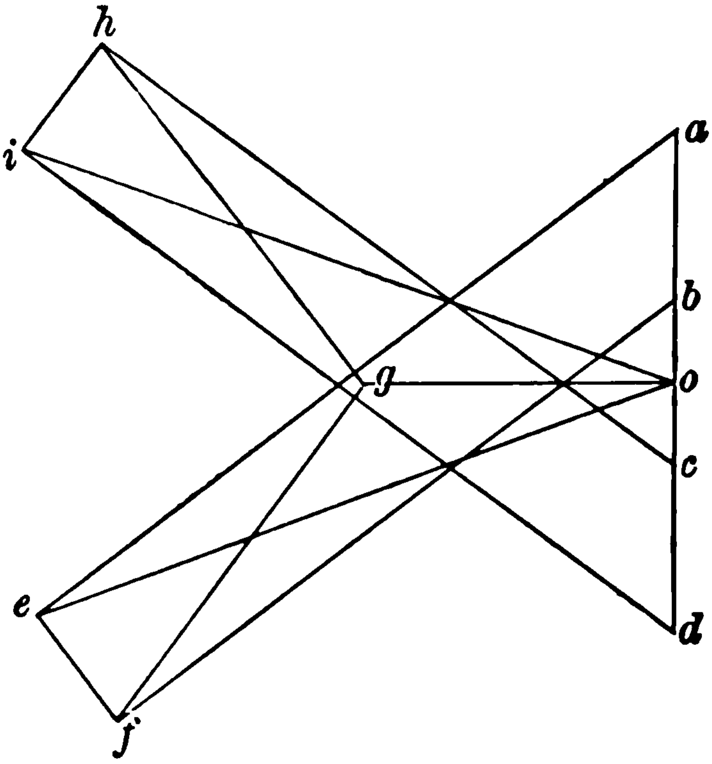


Fig. 6b.

EXAMPLE 7. — Iron truss (Fig. 7), span 80 feet, pitch 30 degrees, distance between trusses from centres, 20 feet.

The loads for the truss with a slate roof on an inch and a quarter board or iron purlins, would be about as indicated on drawing.

To draw the diagram of strains, lay off the loads on a vertical line for one-half of the truss, which would give *ao* (Fig. 7*a*). Then draw *on* parallel to *ON*, and *an* parallel to *AN*, then *bm* parallel to *BM*, and *nm* parallel to *NM*; next draw *ml* parallel to *ML*; then draw *ci* and *dh*, and draw *ih* in line with *nm*; then draw *lk* parallel to *LK*, and *ik* parallel to *IK*. Draw *hg* parallel to *HG*; and, if drawn right, it will pass through *k*. Draw *og* intersecting *hg* at *g*. This will give all the strains in the truss.

It should be noticed that *hg* lies over *kg*, but they should be measured as two separate lines. This form of truss is generally built wholly of iron. The strains are figured in pounds on Fig. 7*a*,

and the size of the various pieces may be computed by the rules for struts and ties. In Fig. 7 the pieces *ML*, *KI*, *KG*, and *HG*, and the principal tie, are all ties; the other pieces being in compression. The piece *GG* is only a light rod which is used to prevent the main tie from sagging.

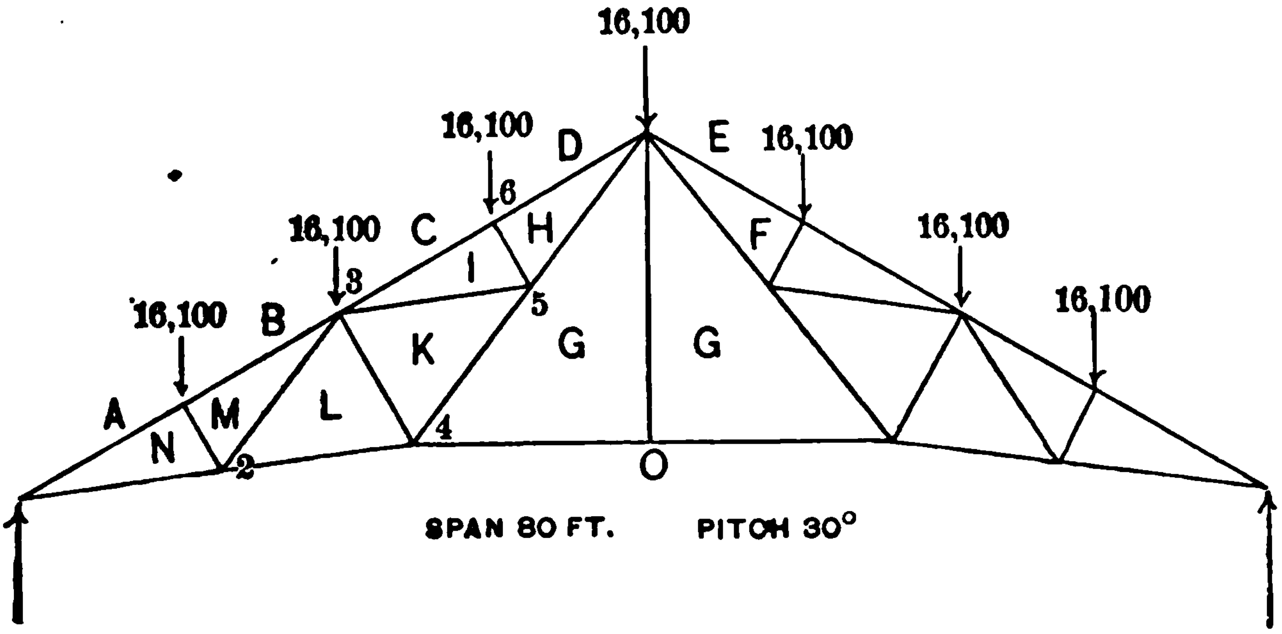


Fig. 7.

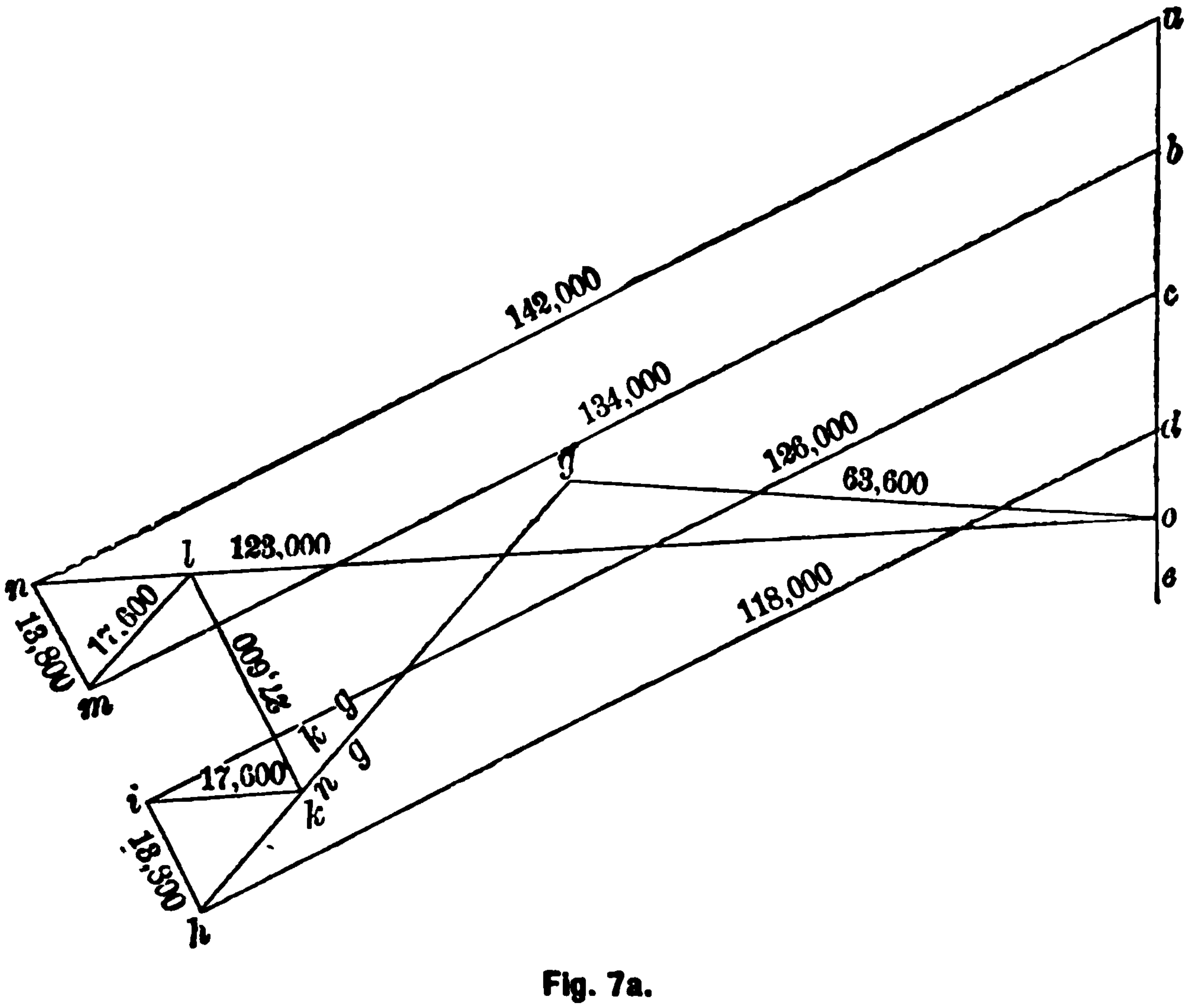


Fig. 7a.

Example 8 (*Iron Bowstring Truss*).—Span of truss, 90 feet; distance between trusses, from centres, 20 feet; rise of arched rafter, 20 feet.

This form of truss, represented by Fig. 8, is one of the most economical of trusses for very great spans.

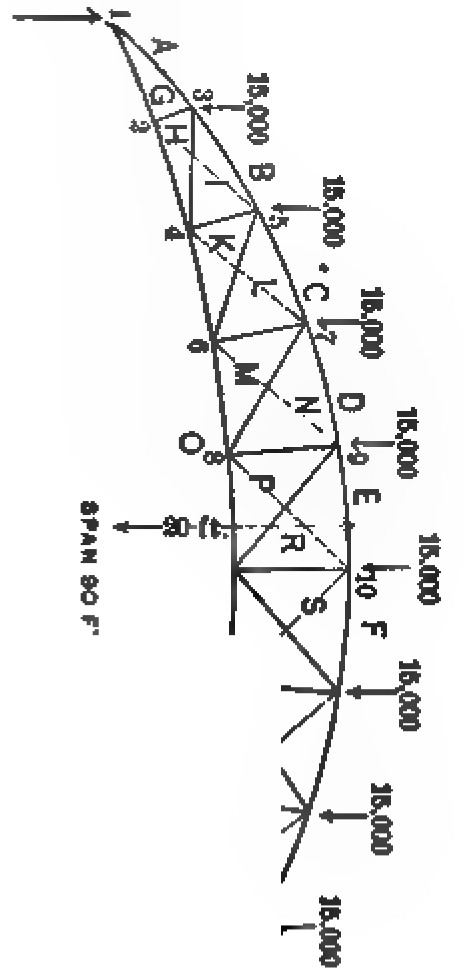


Fig. 8.

In such cases as the present example, the rafter, or curved principal, is the only piece that is in compression, and all the others are in tension. Under a steady dead load only, such as the weight of the roof itself, one set of braces, placed as shown in Fig. 8, would be all that would be needed; but under a severe wind pressure blowing against one side of the roof only, it is necessary to have another set of braces, as shown by the dotted lines in the figure.

These "counter-braces," as they may be called, have no stress on them at all when there is only a vertical load to be supported by the trusses: so we must leave them out in drawing the diagram of strains.

To draw the strain diagram, lay off the loads on a vertical line, as in all the previous examples, and remember that the point *o* should be halfway between *e* and *f* (Fig. 8*a*); then *oa* will be the supporting force at joint 1. In drawing the strains at the different joints, draw first the strains at joint 1, and then at joints 2, 3, 4, 5, etc., in the order in which they are numbered (Fig. 8).

To commence the strain diagram, we have *oa* equal to the supporting force at joint 1, and from *a* draw a line parallel to *AG*, and from *o* a line parallel to *OG*. These two lines intersect at *g*. (In drawing lines parallel to the curved lines of the truss, draw the strain line parallel to a line connecting the two ends of the curved chord. Thus *ag* should be drawn parallel to 1–3, and *og* parallel to 1–2.) At joint 2 we already have *og*, and from *g* draw a line parallel to *GH*, and from *o* a line parallel to *OH* (2–4): this gives the lines *oh* and *gh*.

At joint 3 the strains are *hg*, *ga*, the load *ab*, and the strains *bi* and *hi*.

At joint 4 we now have *oh* and *hi*, and draw *ik* and *ok*. The strains at joints 6 and 8 are drawn in a similar way, and those at 5, 7, and 9, similarly to those at joint 3. After drawing the strains at joint 9, go to joint 10; and, after drawing the strains at that point, all the strains in the truss will have been obtained. The strains in this particular example are given in pounds on the respective lines in the strain diagram. It will be noticed that the strain is very severe on the top and lower chords, but very slight on the bracing. It is in fact so slight, that it will be about as well to make all the diagonal braces of the same size sufficient to resist the strain on *IH*, where the strain is the greatest; or *IH* and *KL* might be the same size, and *MN* and *PR* a smaller size.

The vertical or radiating pieces might be all of a sectional area capable of resisting the strain on *NP*.

The great advantage of this truss lies in the fact that *all its parts*

are in tension, excepting the upper chord, which, of course, is in compression. We might analyze the way in which the strains act, by saying that the upper chord carries all the load, like an arch, and is prevented from spreading out at the ends by the lower tie. The object of the bracing and vertical pieces is only to keep the tie in its curved position, and not allow it to come down flat, and thus allow the ends of the arch to spread out.

EXAMPLE 9 (*The Hammer-Beam Truss*).—As this truss is so frequently used by architects for supporting the roof of churches and large halls, we have devoted considerable space to it.

As generally constructed, hammer-beam roof-trusses exert a more or less horizontal pressure upon the walls supporting them, requiring that the walls shall be heavy, and re-enforced by buttresses on the outside. In churches where the walls are low, this horizontal thrust of the truss is easily taken care of; but in many cases it is desirable to do away with it entirely if possible. In order better to understand the action of the stresses in this truss, we have presented first a truss (Fig. 9) which has all the features of the hammer-beam truss, excepting the lower braces, and yet exerts no horizontal thrust against the wall.

The truss is supposed to be built like the ordinary hammer-beam truss, excepting the omission of the lower braces, and putting in strong timber-ties, *HO* and *PO*, in place of the ornamental curved pieces usually employed. In this particular example we have assumed the span of the truss as 60 feet, the rise as 35 feet, and the distance between centres of trusses 15 feet. This would make the loads at the different joints about as is indicated in Fig. 9.

To draw the strain diagram, lay off the loads on a vertical line in the usual way, the centre coming at *o* (Fig. 9*a*) halfway between *d* and *e*. Now at joint 1 we have the strains *oa*, *af*, and *fo*; at joint 2, *fa*, *ab*, *bg*, and *fg*; at joint 3, *of*, *fg*, *gh*, and *oh*, *oh* acting from *h* to *g*, and hence is a pulling strain. At joint 4 we have *bg*, *gh*, *bc*, *ci*, and *hi* to close the figure: *hi* is also in tension. At joint 5 we have *ic*, *cd*, *dk*, and *ik*. At the top joint 6, the strains are *kd*, *de*, *el*, and *kl*, which completes the strain diagram for one-half of the truss, which, of course, is all that is needed. Examining, now, the diagram, we find that the strains are in general much larger than would be the case if there were a horizontal tie across the truss: still, if we make the pieces large enough to withstand these strains, the truss will be stable, and exert no outward thrust on the walls.

Looking at Fig. 9 we see that *OF*, *H P*, and *R*, form a continuous tie, only it is pulled up in the centre in the form shown. In Fig. 9*a* we see that the strain in the tie-rod *KL* is very great, and

This is because the rod has to hold up the inclined ties HO and PO. If we imagine the tie KL to be cut in two just above

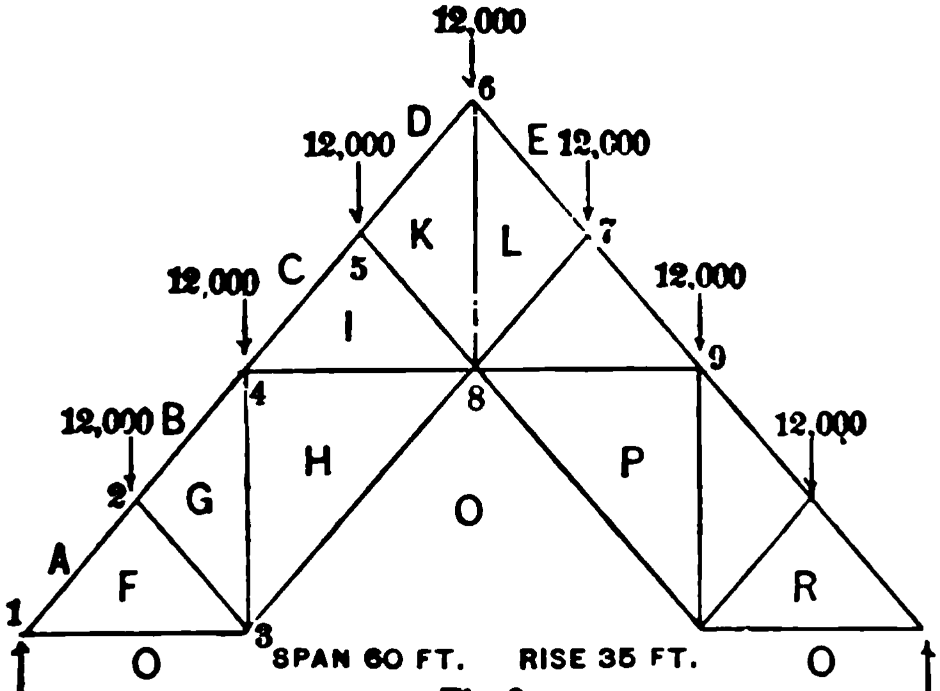


Fig. 9

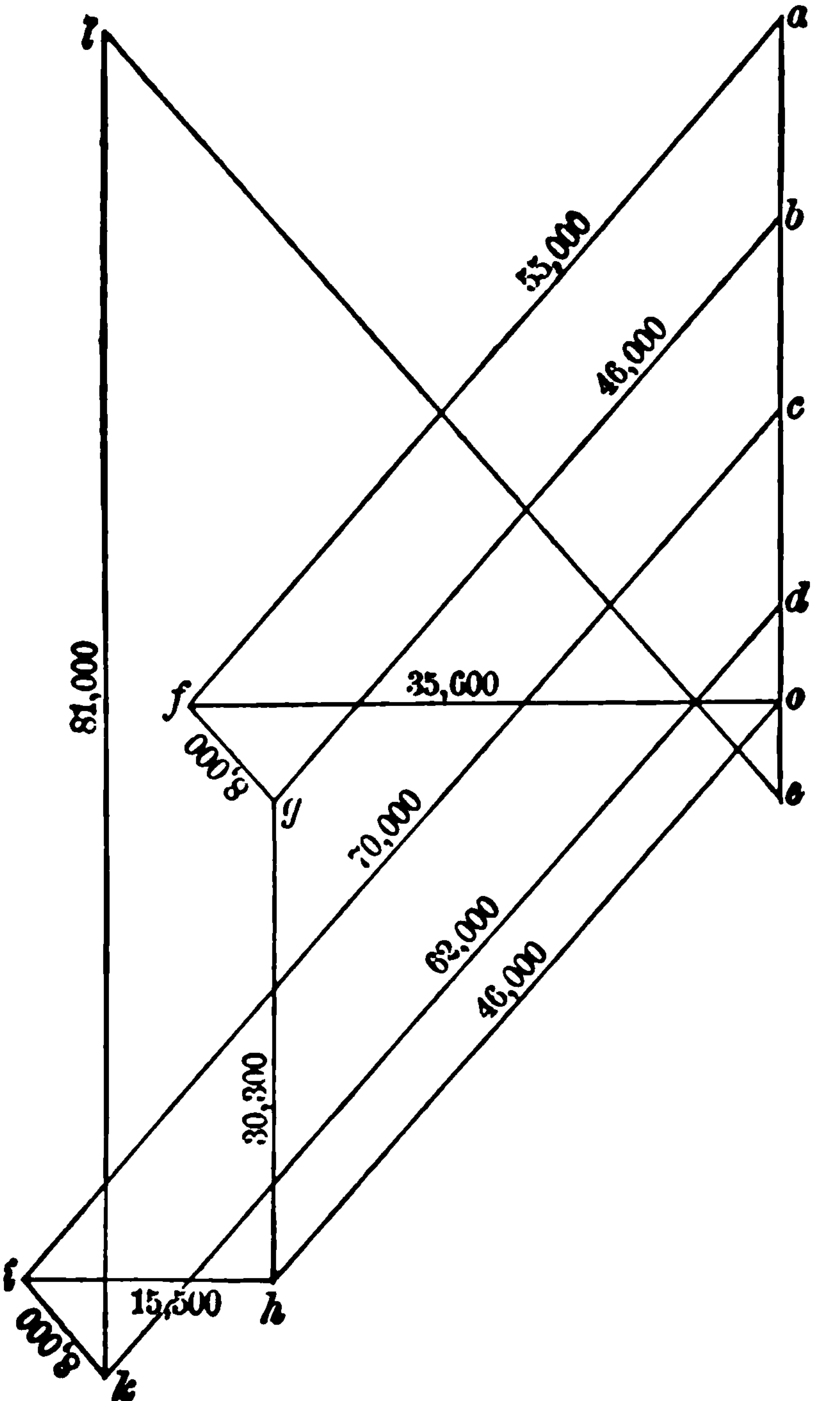


Fig. 1

the joint, the main rafters would break at the joints 4 and 9, and the bottom portion immediately slide outwards, straightening out the main tie, and allowing the top of the truss to fall through.

Having seen that a hammer-beam truss *could* be built in which there is no horizontal thrust, we will now consider the hammer-beam truss as usually built, in which a horizontal thrust is expected. The diagram of such a truss is shown in Fig. 10, in which the curved braces usually built in the centre of the truss are not shown; as they are considered to be purely ornamental, and have no strains in them. The brace *OM* is drawn as though it were straight: but a curved brace can be used as well, without altering the diagram; for the reason that the strain in the curved piece acts in a straight line connecting the centres of each end of the brace.

To draw the strains in this truss we must first find the horizontal thrust of the truss against the wall.

To do this we have to consider that all the piece from joints *o* to joint 4 simply form a framed brace supporting the upper portion of the truss at joint 4, or that a single brace, shown by the dotted line 04, would have the same effect on the wall as all the pieces put together in the framed strut; that is, we may consider the truss to have the same horizontal thrust as the truss shown in Fig. 10*a*. The load at joint 4 would evidently be 12,000 pounds plus load at joint 5, plus half-load at joint 6, and half-load at joint 2; making in all 36,000 pounds. To draw the horizontal thrust and strains we proceed as follows: —

Lay off *ab* (Fig. 10*b*) = load at joint 2, *bc* = load at joint 4, *cd* = load at joint 5, and *de* = load at joint 6. Then the load at joint 4 (Fig. 10*a*) $= \frac{1}{2}\, ab + bc + cd + \frac{1}{2}\, de$; and if we draw from *x* a horizontal line to the left, and from the centre of *ab* a line parallel to 04 (Fig. 10*a*), these two lines will intersect at *i*, and *ix* will be the horizontal thrust exerted on the wall at the point *o*.

Having obtained this thrust, it is easy to determine the strains in the pieces.

At joint *o* we have the thrust *ix*, the vertical supporting force *xa*, and the stresses *ao* and *mo* closing the figure. At joint 1 we have *oa*, *af*, and *of*, as the strains in *OA*, *AF*, and *FO*.

At joint 3 the strains are *mo*, *of*, *fg*, and *mg*; at joint 2 they are *fa*, *ab*, *bg*, and *gf*; at joint 4 the strains are *mg*, *gb*; *bc* and *ci* closing the figure. It will be noticed that the figure closes without allowing any line to be drawn parallel to *MI*: hence there is no tensional strain in *MI*. There must be, however, a compressive strain on *MI* equal to the outward thrust on the walls; but this is not shown in the strain diagram.

At joint 5 we have the strains *ic*, *cd*, *dk*, and *ki*, and at joint 6 we have *kd*, *de*, *el*, and *kl*; which completes the strains for one-half of the truss, which is all we need.

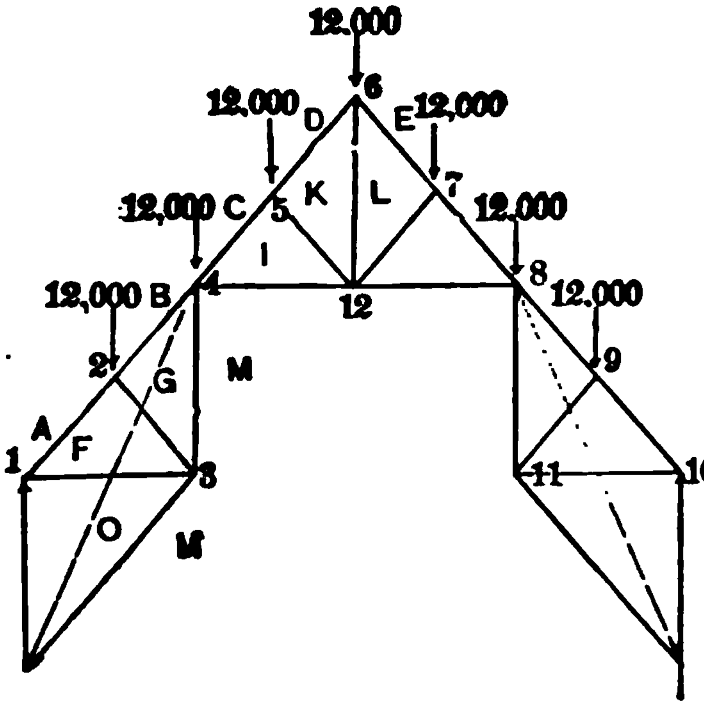


Fig. 10.

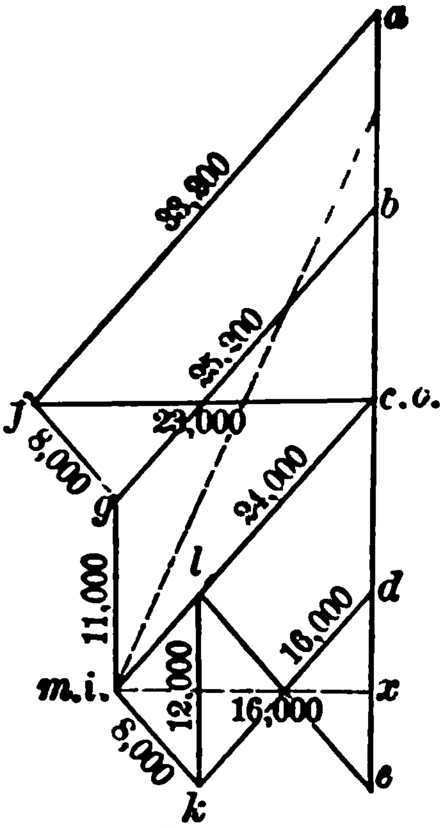


Fig. 10b.

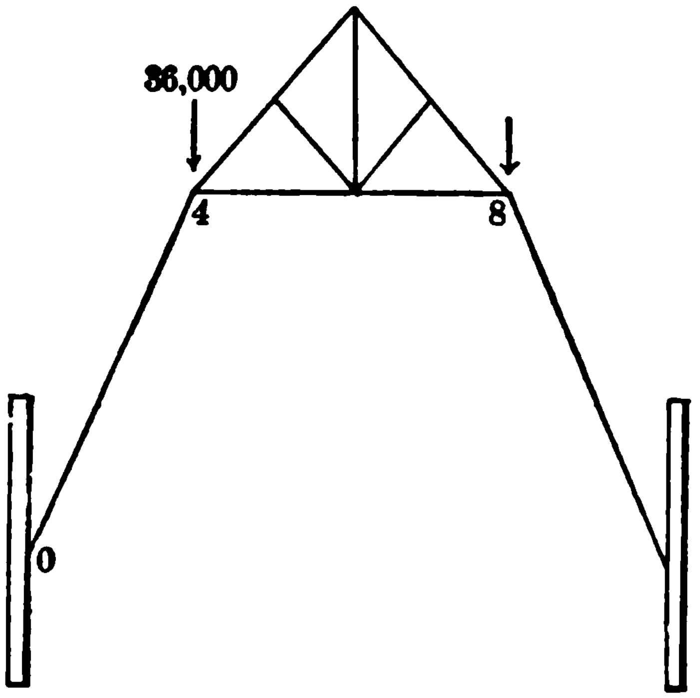


Fig. 10a.

Comparing, now, the diagram of strains, Fig. 10*b*, with Fig. 9*a*, we find that in general the strains in the truss, Fig. 10, are much less than in the truss, Fig. 9; while, on the other hand, the latter truss exerts no outward thrust on the walls, as is the case in Fig. 10.

By building a truss like Fig. 10, and putting in curved ties from joints 3 and 11 to joint 12, we can relieve the brace *OM* of part of

the load without straining the other timbers as much as is the case in Fig. 9.

The truss shown in Fig. 33, Chap. XXVI., combines the advantages of both the forms of hammer-beam trusses which we have considered, though it may not be quite so pleasing to the eye.

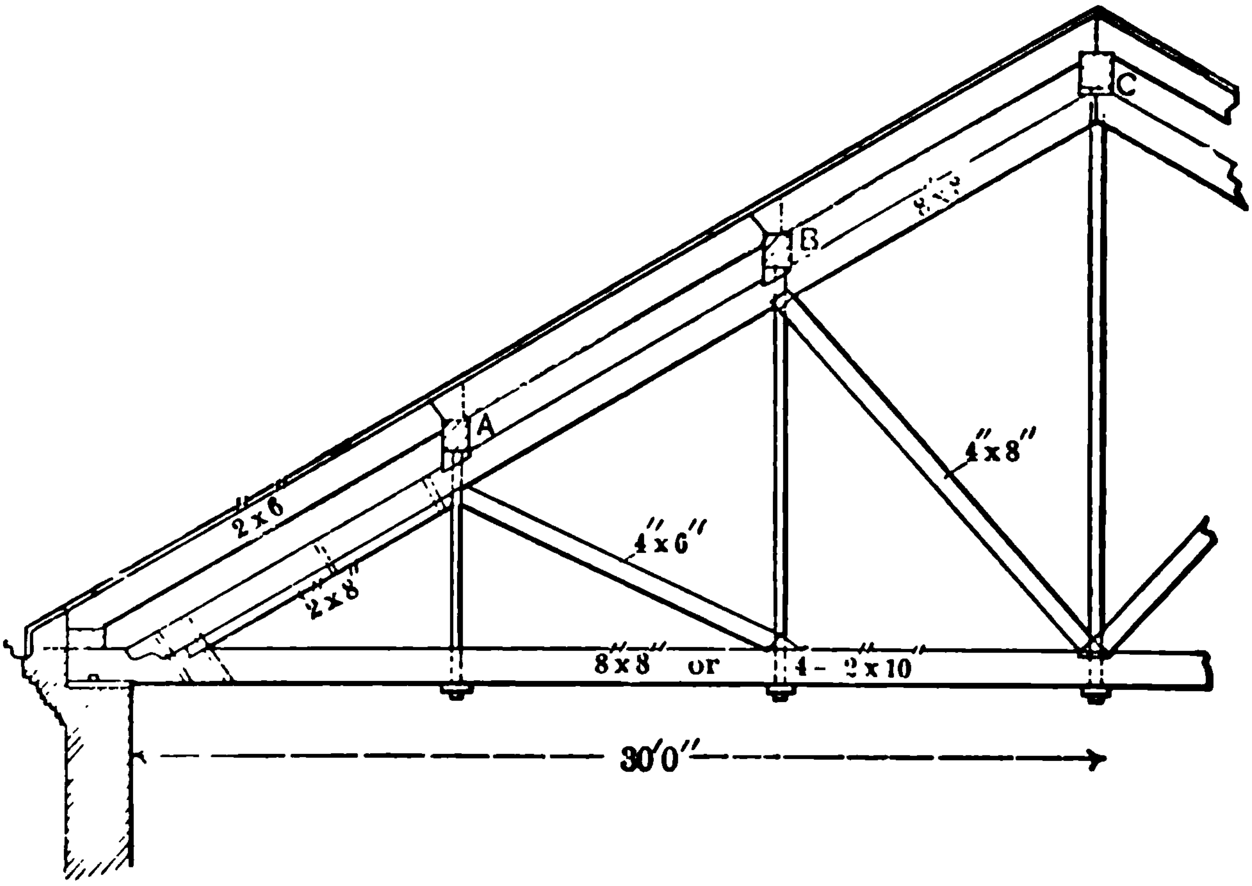


Fig. 11.

EXAMPLE 10 (*Roof of Sixty Feet Span*). — The above nine examples contain all the processes used in determining the strains in any roof-truss under a vertical load; but, in order to render the process of computing the stresses and dimensions of a roof-truss as clear as possible, we have given the whole process of working out the dimensions for a roof over a freight-house or skating-rink in which the width of the building is sixty feet clear span, and the roof makes an angle with the horizon of thirty degrees.

The roofing will consist of an inch and a quarter boards, covered with slate. There will be no ceiling suspended from the trusses; but the roof-boards will be planed so as to show underneath. The trusses will be spaced twenty feet on centres, lengthwise of the building; and there will be two trussed purlins on each side of the roof. The jack-rafters will be two-inch by six-inch spruce plank, planed, spaced twenty inches on centres.

Wooden Roof-Truss. — We will compute the stresses and dimensions, first for a timber truss of hard pine, and then for an iron truss, the jack-rafters and roofing being the same in each instance. Fig. 11 shows the best form of a wooden truss to meet the required conditions, and we will at once proceed to deter-

mine the stresses in the different parts of the truss. To do this, we must first determine the loads to be supported at the points A, B, and C, where the purlins rest on the truss. The distance between the centres of the purlins we find by our scale to be 11 feet and 6 inches; and, as the trusses are twenty feet apart, each purlin will support a roof area of $11\frac{1}{2} \times 20$ feet = 230 square feet.

The weight per square foot of roof, we find from the preceding tables will be, —

For slate on $1\frac{1}{4}$-inch boards	16 pounds.
wind pressure, angle of 30°	26 "
snow	15 "
Total weight per square foot	57 pounds.

The total load carried by purlins will then be $230 \times 57 = 13{,}110$ pounds; or the load coming on the truss at each of the points A, B, and C, will be, say 13,100 pounds. And this is all the load to be provided for; as there is no ceiling, and nothing suspended from the truss, and the weight of the truss itself is included in the weight of the slate.

We now proceed to draw a diagram of the truss, representing the line passing through the centre of each piece, to an accurate scale, as in Fig. 12, and are then ready to draw our diagram of strains, as in Fig. 12*a*.

To draw this diagram, we first lay off to a scale, on a vertical line; the loads bc, cg, and gh, representing the loads on the truss at the joints 2, 4, and 5. Bisect gh at the point o, and the line ob will represent the load on one-half of the truss.

Now draw ab and ao (Fig. 12*a*.) parallel to AB and AO (Fig. 12), and they will represent the strains at the joint 1. At joint 2 we know the strain ab, the load bc; and we draw cd parallel to CD, and ad parallel to AD, which gives us all the strains at joint 2.[1]

At joint 3 we have the strains ao and ad; and we draw de, and we then have the stresses in DE and EO.

At joint 4 we have the strains ed and dc, and the load cg, and have only to draw gf parallel to GF, and ef parallel to EF, to complete the strain diagram for that joint.

Lastly, going to joint 5, we have the strain gf and the load gh, and drawing hi parallel to HI, and fi parallel to FI, we have our

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[1] The tie AA (Fig. 12) cannot be represented in the strain diagram, for the reason that there is no strain upon the rod at all, coming from the load on the truss; and the only use of the rod is to keep the tie AO from sagging: hence the strain diagram should be made the same as though the rod AA were not in the truss at all.

strain diagram complete for one-half of the truss; and that is all we require, as the stresses in both sides of the truss will be the same.

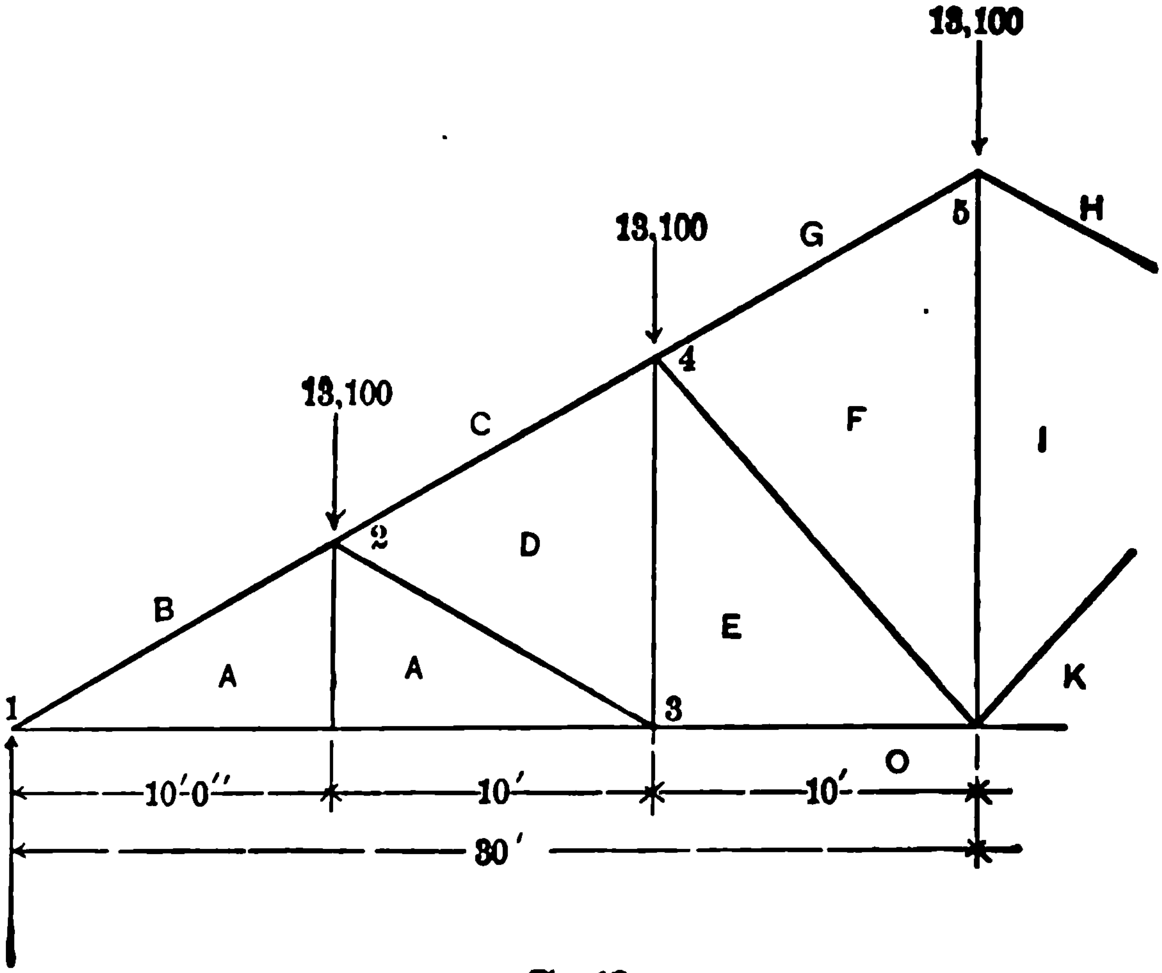

Fig. 12.

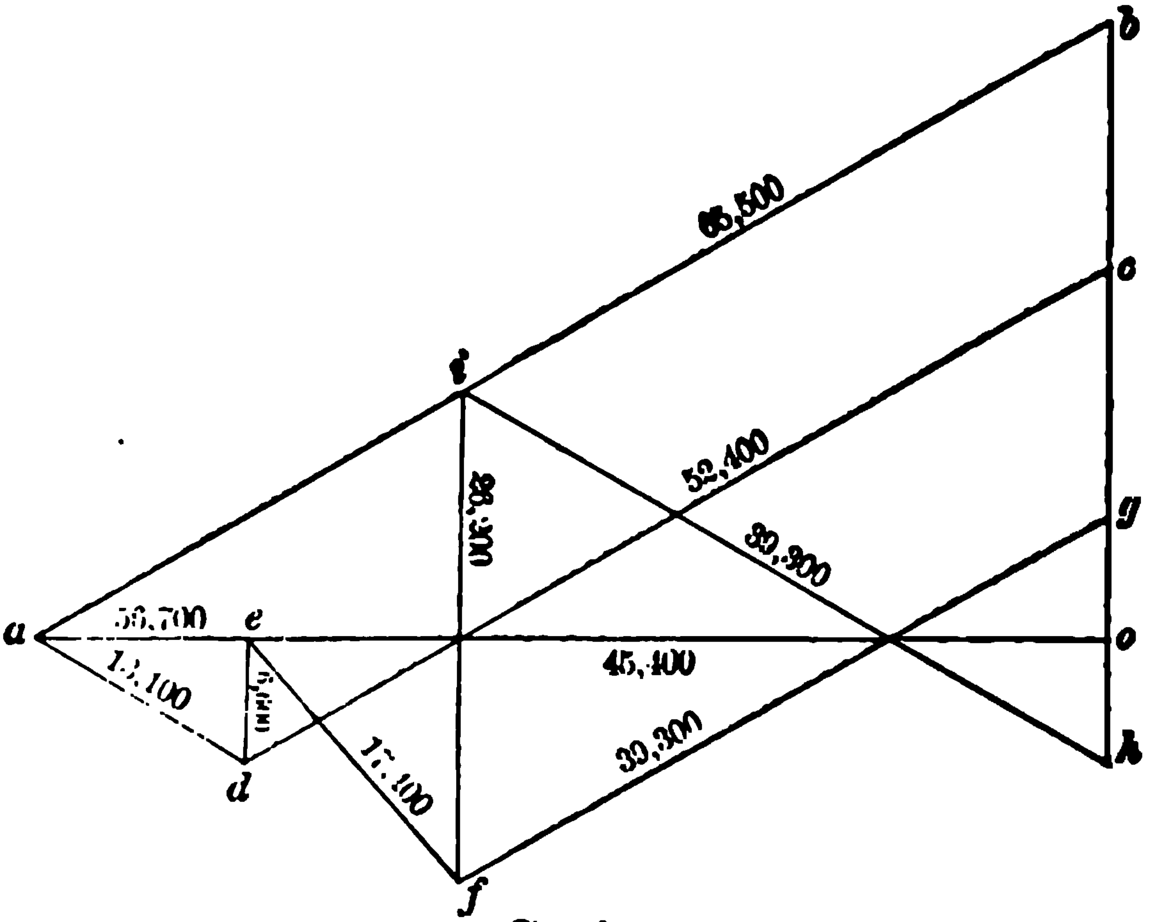

Fig. 12a.

Applying, now, our scale to the lines in the diagram of strains (Fig. 12*a*), we find the stress in the rafter *AB* to be 65,500 pounds, as figured on the line *ab*, the stress in *AD* to be 13,100 pounds, as

figured on *ad*, and the stresses in the other pieces to be as shown by the figures above the corresponding lines in the strain diagram; and this brings us to the last step in the problem, — to proportion the various parts of the truss to their respective stresses.

The Rafters. — The greatest stress in the principal rafter of the truss is in the section *AB*, which has a compressive stress of 65,500 pounds. As the length of the section is only 11 feet and 6 inches, we may safely allow 1000 pounds per square inch of cross-section as the working-stress in the rafter, which would give us 65½ square inches of cross-section as the required area. As the timbers of the truss will be planed, it will be hardly safe to use an 8 by 8 timber; and, as the next largest merchant size is 8 by 10, we will use that size.

The stress in the section of the rafter *CD* is 52,400 pounds, and for this an 8-inch by 8-inch timber will be more than strong enough. As the stress in *GF* is still less, we will make the whole rafter of one piece of eight-inch by eight-inch timber, with a two-inch by eight-inch plank bolted to the under side of it in the lower section, as shown in Fig. 11.

Braces. — The stress in the brace or strut *AD* is 13,100 pounds; and for this we will use a four-inch by six-inch timber, a three-inch plank being liable to spring for so long a length.

The strut *EF* has only 17,400 pounds' stress on it; but, being so long, we will use a four-inch by eight-inch timber for it.

Tie-Beam. — The maximum strain in the tie-beam is 56,700 pounds; and, as hard pine may safely be trusted with 2000 pounds per square inch tensile strain, we need only have 28 square inches of timber in the least cross-section of the tie-beam; but as we shall have to cut into it some, and the rods must go through it, and the beam should be as wide as the struts and rafters in order to make a good joint, we will make the tie-beam of one piece of eight-inch by eight-inch hard pine. If it is found impracticable to get a timber sixty-three feet long of that size, we could use two-inch by ten-inch plank bolted together so as to break joint, and make a beam eight inches by ten inches.

Rods. — The rod *AA* need only be a half-inch rod, as it is only to keep the tie *AO* from sagging. The rod *DE* has a tensile stress of 6600 pounds to resist; and, as wrought-iron has a safe resistance of 10,000 pounds to the square inch, we need about 0.66 square inch of cross-section in the rod. A rod seven-eighths of an inch in diameter has an area of 0.60 inch, and an inch rod, 0.78 inch: hence we must use an inch rod with the ends upset, or an inch and a quarter rod if the ends are not upset. For the rod *FI*, we need 2.6 square inches of cross-section, which requires a rod an inch and

seven-eighths in diameter if the screw-end is upset, and two and one-fourth inches if the end is not upset.

Thus we have determined the dimension of each piece of our truss, and may feel sure that there will be no danger of its falling down as long as the timber remains sound.

The Purlins. — Having decided upon the proportions of our truss, we will now decide what we will use for the purlins. To give a light appearance to the roof, and also keep it good and stiff, without sagging between the trusses, we will use a trussed purlin, like that shown in Fig. 13. The load upon each purlin we have already found to be 13,100 pounds; and it can be proved, that, with a beam supported at four points, the load coming on each of the two middle points of support will be 0.367 of the whole weight on the beam. Then, denoting the weight over one of the struts *S* by W'_1, its value would be $0.367 \times 13{,}100 = 4807$ pounds, or, for practical purposes, 4800 pounds.

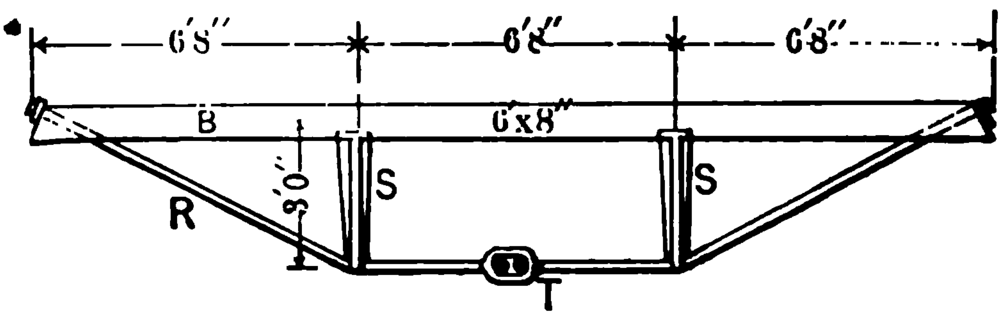

Fig. 13.

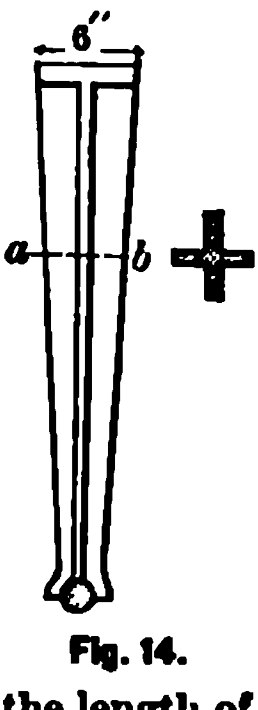

Fig. 14.

Now, the strain in the tie *R* is to the strain on *S* as the length of *R* is to the height of the truss from centre of rod to centre of beam. We will make this height 3 feet, and we find the length of *R*, by our scale, to be 7 feet 4 inches; then,

$$\text{Strain in } R : 4800 \text{ pounds} :: 7\tfrac{1}{3} : 3,$$

or

$$\text{Strain in } R = \frac{7\frac{1}{3} \times 4800}{3} = 11{,}733 \text{ pounds.}$$

This would require a rod an inch and one-fourth in diameter with the screw-ends upset. The rod should have a turn-buckle at T.

The beam *B* would have a compressive strain $= \dfrac{6\frac{2}{3} \times 4800}{3} =$ 10,666 pounds, which would require a beam about 1½ inch by 8 inches; but, as the beam has also to carry the weight of the jack-

rafters between two points of support, we shall be obliged to use a six-inch by eight-inch timber for the straining-beam of our purlin.

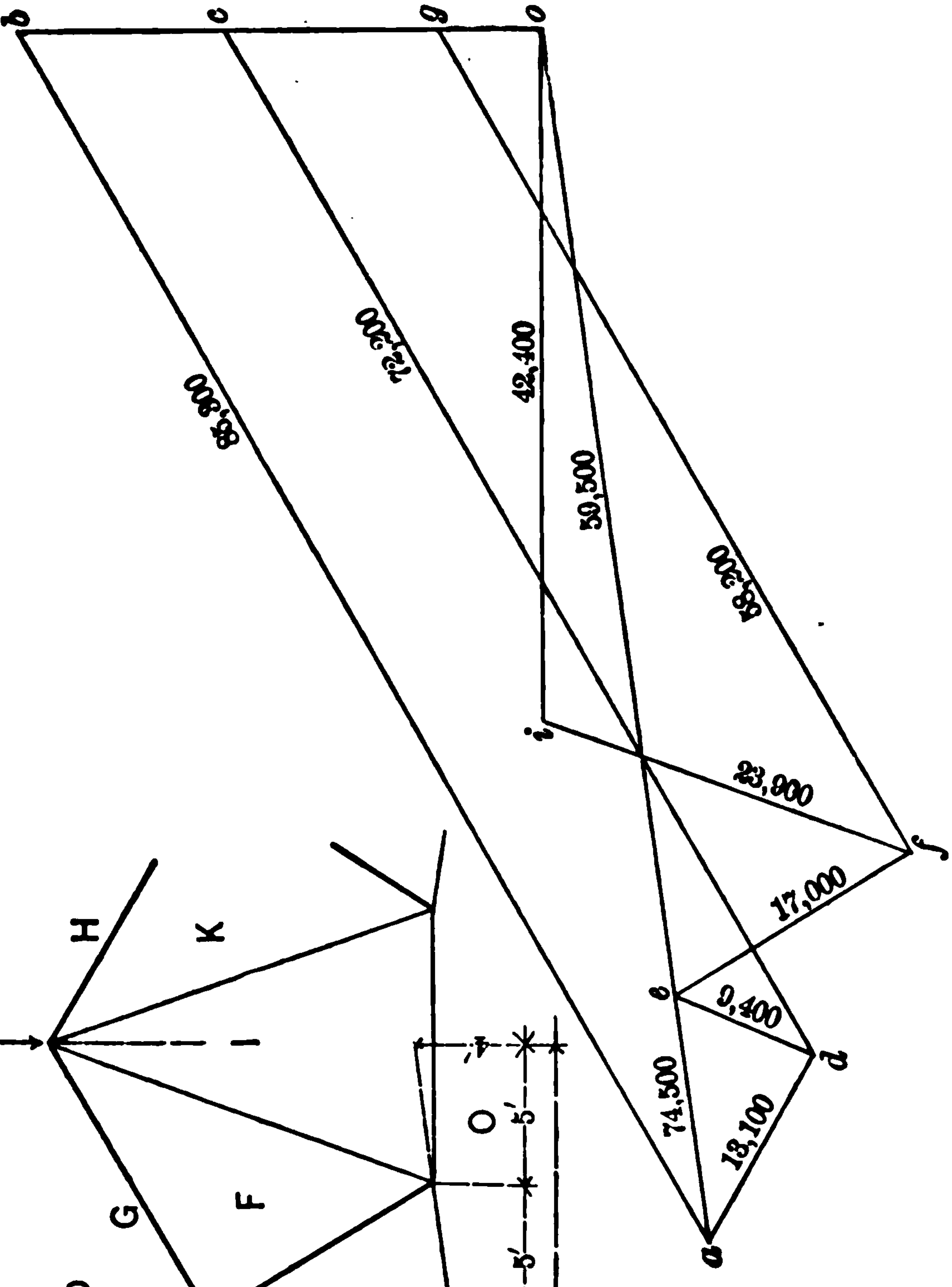

Fig. 15a.

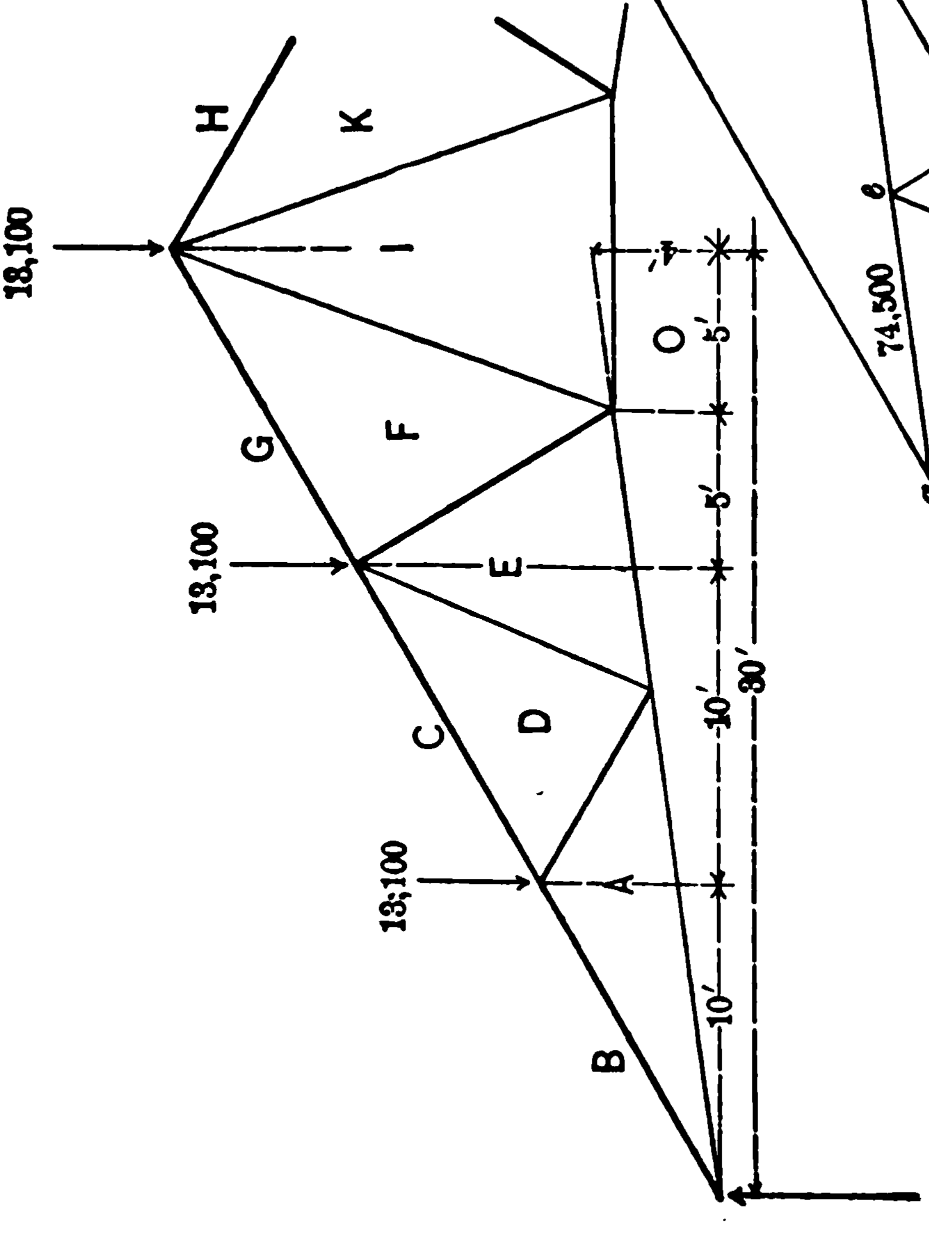

Fig. 15.

are in tension, excepting the upper chord, which, of course, is in compression. We might analyze the way in which the strains act, by saying that the upper chord carries all the load, like an arch, and is prevented from spreading out at the ends by the lower tie. The object of the bracing and vertical pieces is only to keep the tie in its curved position, and not allow it to come down flat, and thus allow the ends of the arch to spread out.

EXAMPLE 9 (*The Hammer-Beam Truss*).—As this truss is so frequently used by architects for supporting the roof of churches and large halls, we have devoted considerable space to it.

As generally constructed, hammer-beam roof-trusses exert a more or less horizontal pressure upon the walls supporting them, requiring that the walls shall be heavy, and re-enforced by buttresses on the outside. In churches where the walls are low, this horizontal thrust of the truss is easily taken care of; but in many cases it is desirable to do away with it entirely if possible. In order better to understand the action of the stresses in this truss, we have presented first a truss (Fig. 9) which has all the features of the hammer-beam truss, excepting the lower braces, and yet exerts no horizontal thrust against the wall.

The truss is supposed to be built like the ordinary hammer-beam truss, excepting the omission of the lower braces, and putting in strong timber-ties, *HO* and *PO*, in place of the ornamental curved pieces usually employed. In this particular example we have assumed the span of the truss as 60 feet, the rise as 35 feet, and the distance between centres of trusses 15 feet. This would make the loads at the different joints about as is indicated in Fig. 9.

To draw the strain diagram, lay off the loads on a vertical line in the usual way, the centre coming at *o* (Fig. 9*a*) halfway between *d* and *e*. Now at joint 1 we have the strains *oa*, *af*, and *fo*; at joint 2, *fa*, *ab*, *bg*, and *fg*; at joint 3, *of*, *fg*, *gh*, and *oh*, *oh* acting from *h* to *g*, and hence is a pulling strain. At joint 4 we have *bg*, *gh*, *bc*, *ci*, and *hi* to close the figure: *hi* is also in tension. At joint 5 we have *ic*, *cd*, *dk*, and *ik*. At the top joint 6, the strains are *kd*, *de*, *el*, and *kl*, which completes the strain diagram for one-half of the truss, which, of course, is all that is needed. Examining, now, the diagram, we find that the strains are in general much larger than would be the case if there were a horizontal tie across the truss, still, if we make the pieces large enough to withstand these strains, the truss will be stable, and exert no outward thrust on the walls.

Looking at Fig. 9 we see that *OF*, *H*, *P*, and *R*, form a continuous tie, only it is pulled up in the centre in the form shown. In Fig. 9*a* we see that the strain in the tie-rod *KL* is very great, and

his is because the rod has to hold up the inclined ties *HO* and *PO*. If we imagine the tie *KL* to be cut in two just above

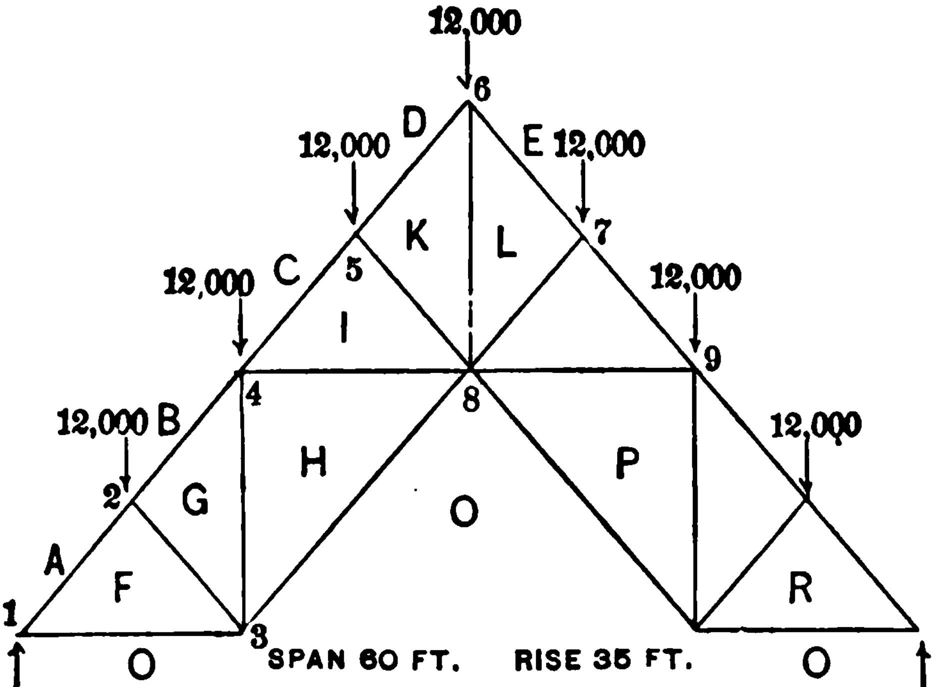

Fig. 9

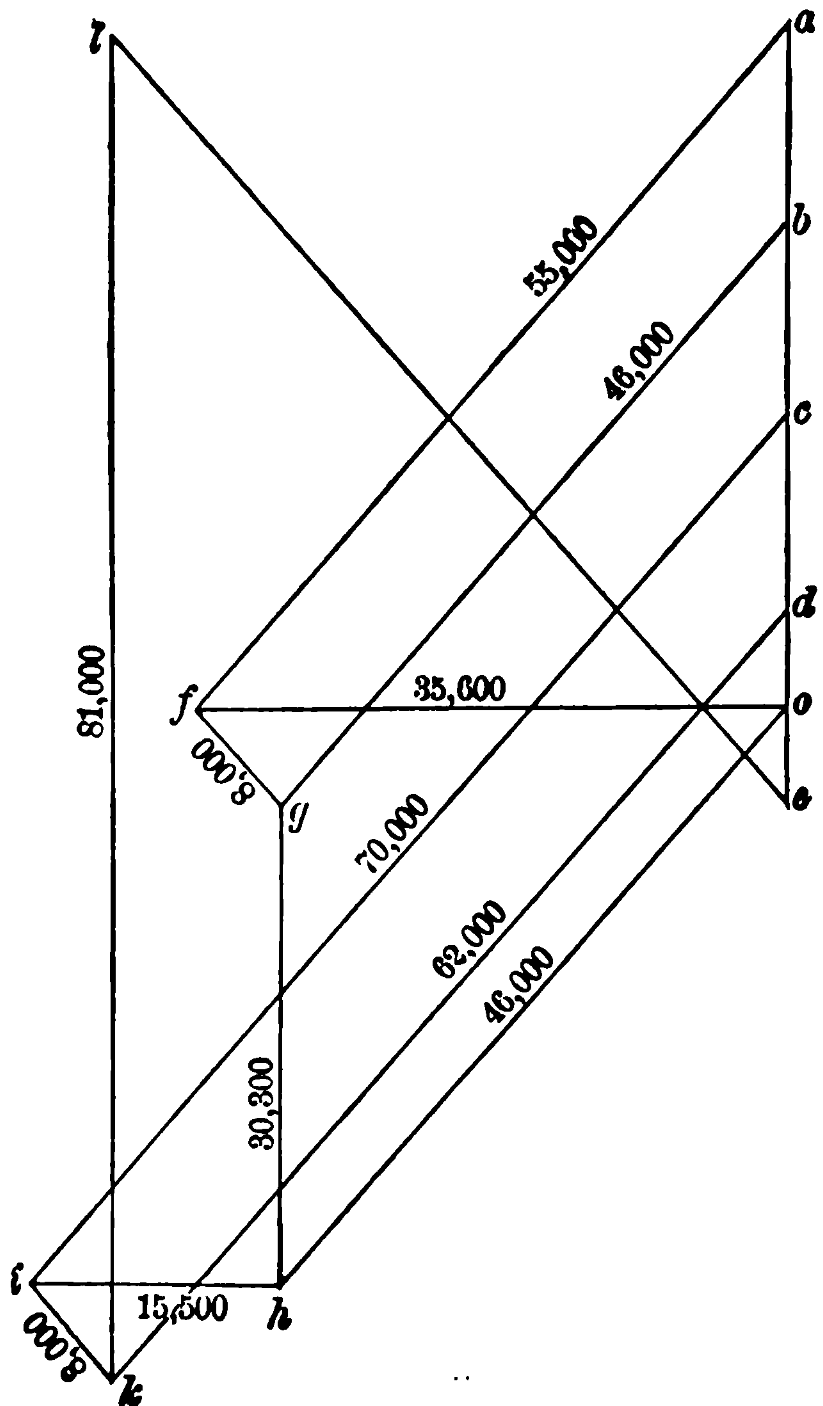

Fig. 9a.

CHAPTER XXIX.

JOINTS.

THE stability of any piece of frame-work depends in a very great measure upon the manner in which the joints are made. It is therefore very important, that in drawing trusses, or frame-work of any kind, the designer should have a good knowledge of the fundamental principles upon which every joint should be constructed, and of the most approved methods of forming the principal joints found in frame-work.[1]

Joints are the surfaces at which the pieces of a frame touch each other. They are of various kinds, according to the relative positions of the pieces and to the forces which the pieces exert on each other.

Joints should be made so as to give the largest bearing-surfaces consistent with the best form for resisting the particular strains which they have to support, and particular attention should be paid to the effects of contraction and expansion in the material of which they are made.

In planning them the purpose they are to serve must be kept in mind, for the joint most suitable in one case would oftentimes be the least suitable in another.

JOINTS IN TIMBER-WORK.

In frames made of timber, the pieces may be joined together in three ways, — by connecting them;

1. End to end;

[1] As the author could think of no better way in which to present the subject, he has taken, by permission of Professor Wheeler and of the publishers, the following chapter on Joints from the text-book, on Civil Engineering, prepared by Professor Wheeler for the use of the cadets of the United-States Military Academy, and published by John Wiley & Sons of New York. The author heartily recommends Professor Wheeler's work to the architect or builder who wishes to obtain a thorough knowledge of construction and the materials employed therein.

2. The end of one piece resting upon or notched into the face of another; and

3. The faces resting on, or notched into each other.

I. Joints of Beams united End to End, the axes of the beams being in the same straight line.

First, Suppose the pieces are required to resist strains in the direction of their length.

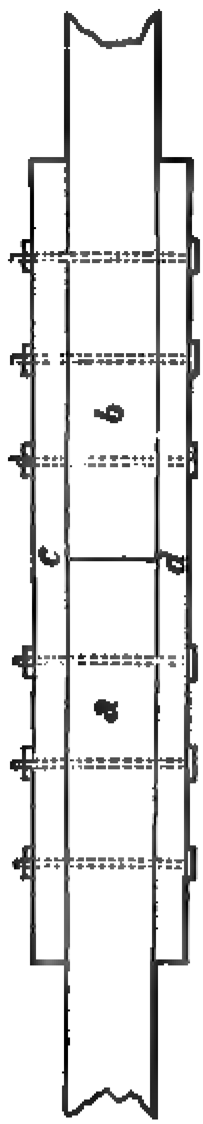

Fig. 1.
Represents the manner in which two beams, *a* and *b*, are fished by side-pieces, *c* and *d*.

Fig. 2.
Represents a joint to resist extension, iron rods or bars being used to connect the beams, instead of wooden fish-pieces.

Fig. 3.
Represents a fished joint in which the side-pieces *c* and *d* are either let into the beams, or secured by keys *e*, *e*.

This case occurs, when, in large or long frames, a single piece of the required length cannot be easily procured.

The usual method of lengthening is in this case by *fishing* or *scarfing*, or by a combination of the two.

Fish-Joints. — When the beams abut end to end, and are connected by pieces of wood or iron placed on each side, and firmly

bolted to the timbers, the joint is called *a fish-joint*, and the beam is said to be *fished*.

This joint is shown in Fig. 1, and makes a strong and simple connection.

When the beams are used to resist a strain of compression, the fish-pieces should be placed on all four sides, so as to prevent any lateral movement whatever of the beams.

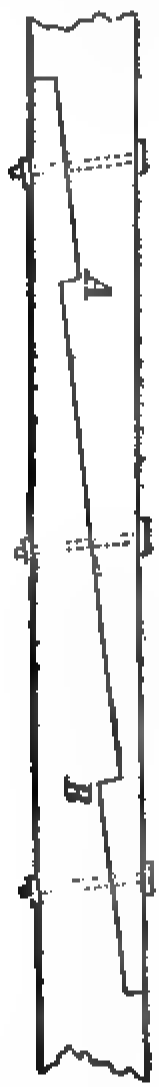

Fig. 5.

Fig. 4.

If the strain be one of tension, it is evident that the strength of the joint depends principally upon the strength of the bolts, assisted by the friction of the fish pieces against the sides of the timber.

The dependence upon the bolts may be much lessened by notching the fish-pieces upon the beams, as shown on the upper side of the piece in Fig. 3 ; or by making use of keys or blocks of hard wood inserted in shallow notches made in both the beam and fish-piece, as shown on the lower side of the piece in the same figure.

Care should be taken not to place the bolts too near the ends of the pieces. The sum of the areas of cross-sections of the bolts should not be less than one-fifth that of the beam.

Scarf-Joints.—In these joints the pieces overlap each other, and are bolted together. The form of lap depends upon the kind of strain to which the beam is to be subjected.

Fig. 6.

Represents a scarf-joint secured by iron fish-plates, *c*, *c*, keys, *d*, *d*, and bolts.

Fig. 7.

Represents a scarf-joint for a cross-strain, fished at bottom by a piece of timber *c*.

Fig. 4 is an example of a simple scarf-joint that is sometimes used when the beam is to be subjected only to a slight strain of extension. A key or folding wedge is frequently added, notched equally in both beams at the middle: it serves to bring the surfaces of the joint tightly together.

This joint is often made by cutting the beams in such a manner as to form projections which fit into corresponding indentations. A good example, in which two of these notches are made is shown in Fig. 5.

The total lap shown in this figure is ten times the thickness of the timber, and the depth of the notches at A and B are each equal to one-fourth that of the beam. The bolts are placed at right angles to the principal lines of the joint.

This is a good joint where a strain of tension of great intensity is to be resisted, as, by the notches at A and B, one-half of the cross-section of the beam resists the tensile strain.

Combination of Fish and Scarf Joints.—The joint shown in Fig. 6 is a combination of the fish and scarf joints, and is much used to resist a tensile strain.

Second, Suppose the pieces are required to resist a transverse strain.

In this case the scarf-joint is the one generally used, and it is then formed sometimes by simply halving the beams near their ends, as shown in Fig. 6.

The more usual and the better form of joint for this case is shown in Fig. 7.

In the upper portion of this joint the abutting surfaces are perpendicular to the length of the beam, and extend to a depth of at least one-third, and not exceeding one-half, that of the beam. In the bottom portion they extend one-third of the depth, and are perpendicular to the oblique portion joining the upper and lower ones.

The lower side of the beam is fished by a piece of wood or iron plate secured by bolts or iron hoops, so as to better resist the tensile strain to which this portion of the beam is subjected.

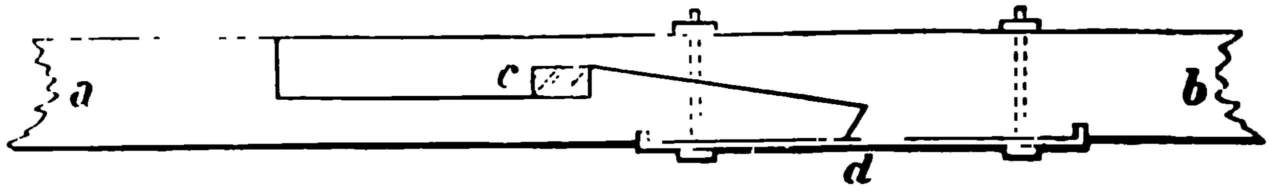

Fig. 8.

Represents a scarf-joint arranged to resist a cross-strain and one of extension. The bottom of the joint is fished by an iron plate; and a folding wedge inserted at c serves to bring all the surfaces of the joint to their bearings.

Third, Suppose the piece required to resist cross-strains combined with a tensile strain.

The joint frequently used in this case is shown in Fig. 8.

In the previous cases the axes were regarded as being in the same straight line. If it be required to unite the ends, and have the axes make an angle with each other, this may be done by halving the beams at the ends, or by cutting a mortise in the centre of one, shaping the end of the other to fit, and fastening the ends together

by pins, bolts, straps, or other devices. The joints used in the latter case are termed "mortise" and "tenon joints." Their form will depend upon the angle between the axes of the beams.

II. **Joints of Beams,** the axes of the beams making an angle with each other.

Mortise and Tenon Joints.—When the axes are perpendicular to each other, the mortise is cut in the face of one of the beams, and the end of the other beam is shaped into a tenon to fit the mortise, as shown in Fig. 9.

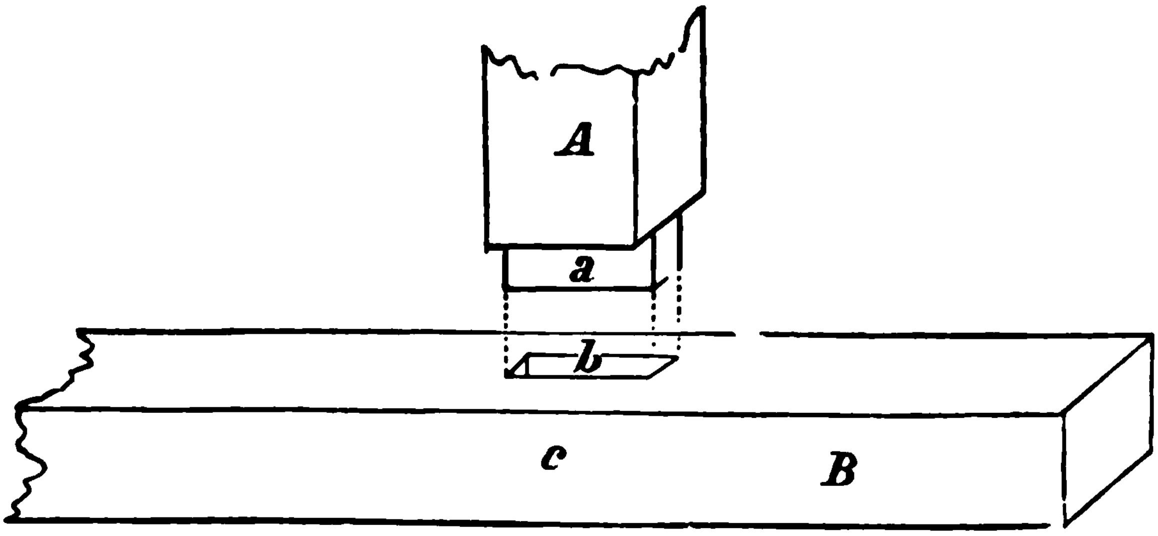

Fig. 9.

Represents a mortise and tenon joint when the axes of the beams are perpendicular to each other. *a*, tenon on the beam *A*; *b*, mortise in the beam *B*; *c*, pin to hold the parts together.

When the axes are oblique to each other, one of the most common joints consists of a triangular notch cut in the face of one of the beams, with a shallow mortise cut in the bottom of the notch, the end of the other beam being cut to fit the notch and mortise, as shown in Fig. 10.

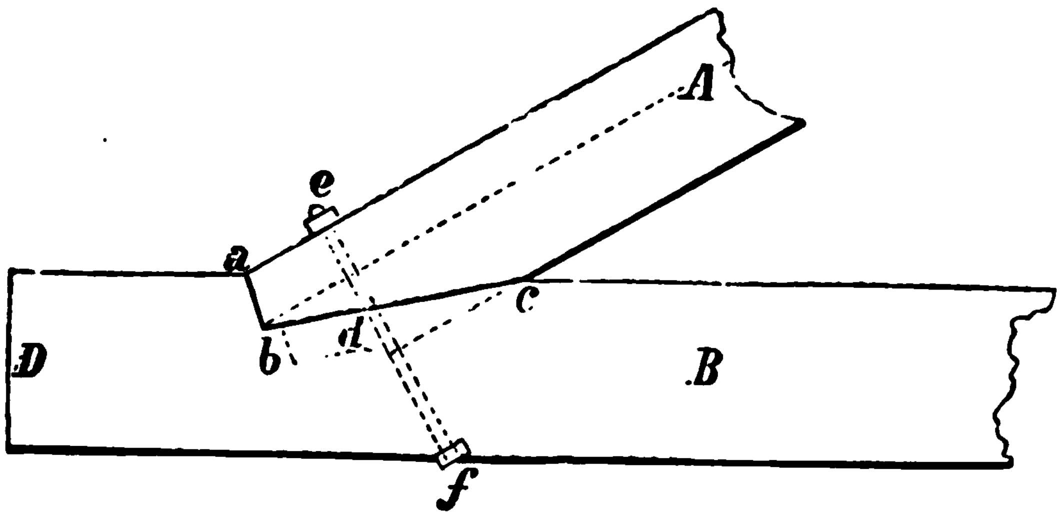

Fig. 10.

Represents a mortise and tenon joint when the axes of the beams are oblique to each other.

In a joint like this the distance *ab* should not be less than one-half the depth of the beam *A*; the sides *ab* and *bc* should be per-

pendicular to each other when practicable; and the thickness of the tenon *d* should be about one-fifth of that of the beam *A*. The joint should be left a little open at *c* to allow for settling of the frame. The distance from *b* to the end *D* of the beam should be sufficiently great to resist safely the longitudinal shearing-strain caused by the thrust of the beam *A* against the mortise.

Suppose the axes of the beams to be horizontal, and the beam *A* to be subjected to a cross-strain; the circumstances being such that the end of the beam *A* is to be connected with the face of the other beam *B*.

In this case a mortise and tenon joint is used, but modified in form from those just shown.

To weaken the main or supporting beam as little as possible, the mortise should be cut near the middle of its depth; that is, the centre of the mortise should be at or near the neutral axis. In order that the tenon should have the greatest strength, it should be at or near the under side of the joint.

Since both of these conditions cannot be combined in the same joint, a modification of both is used as shown in Fig. 11.

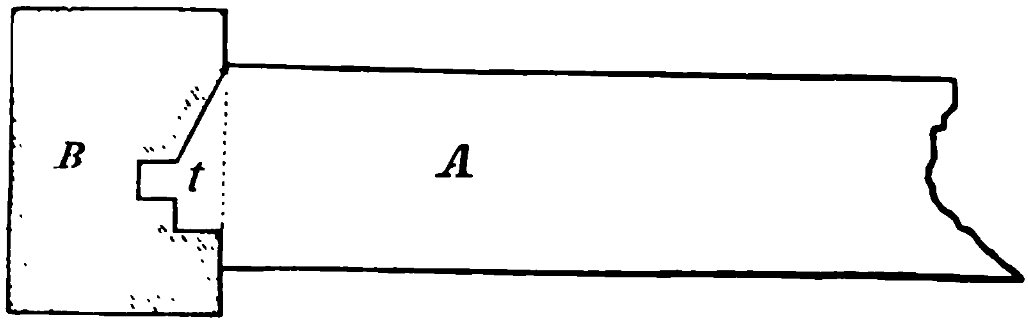

Fig. 11.

A, the cross-beam; *B*, cross-section of main beam; *t*, the tenon.

The tenon has a depth of one-sixth that of the cross-beam *A*, and a length of twice this, or of one-third the depth of the beam. The lower side of the cross-beam is made into a shoulder, which is let into the main beam one-half the length of the tenon.

Double tenons have been considerably used in carpentry. As a rule, they should *never* be used, as both are seldom in bearing at the same time.

III. **Joints used to connect Beams, the Faces resting on or notched into Each Other.**—The simplest and strongest joint in this case is made by cutting a notch in one or both beams, and fastening the fitted beams together.

If the beams do not cross, but have the end of one to rest upon the other, a *dove-tail* joint is sometimes used. In this joint, a notch, trapezoidal in form, is cut in the supporting beam, and the end of the other beam is fitted into this notch.

On account of the shrinkage of timber, the dove-tail joint should never be used, except in cases where the shrinkage in the different parts counteract each other.

It is a joint much used in joiner's work.

The joints used in timber-work are generally composed of plane surfaces. Curved ones have been recommended for struts, but the experiments of Hodgkinson would hardly justify their use. The simplest forms are, as a rule, the best, as they afford the easiest means of fitting the parts together.

Fastenings. — The pieces of a frame are held together at the joints by fastenings, which may be classed as follows: —

1. **Pins,** including nails, spikes, screws, bolts, and wedges;
2. **Straps and tie-bars,** including stirrups, suspending-rods, etc.; and
3. **Sockets.**

These are so well known that a description of them is unnecessary.

General Rules to be observed in the Construction of Joints.

In planning and executing joints and fastenings the following general principles should be kept in view: —

I. To arrange the joints and fastenings so as to weaken as little as possible the pieces which are to be connected.

II. In a joint subjected to compression to place the abutting surfaces as nearly as possible perpendicular to the direction of the strain.

III. To give to such joints as great a surface as practicable.

IV. To proportion the fastenings so that they will be equal in strength to the pieces they connect.

V. To place the fastenings so that there shall be no danger of the joint giving way by the fastenings shearing, or crushing the timber.

RIVETED JOINTS.

The most common method of uniting pieces of wrought-iron or steel in framed structures is by means of rivets. And that the structure shall be equally strong in all its parts, it is essential that the joints shall be carefully designed.

A rivet is a piece of metal with a solid head at one end, and a long circular shank.

Riveting consists of heating the rivet, passing it through the holes in the plates to be united while hot, and then forging another solid head out of the projecting end of the shank.

The hammering causes the heated shank to fill all parts of the holes, and the contraction of the metal, as it cools, draws the heads together, thus firmly forcing and holding the pieces together.

Rivets are generally made either of mild steel or the best wrought-iron, the latter being the most reliable. The rivet-heads are made in four ways, as shown in Fig. 1.

The first shape is the one generally used. The second and third are used only for their appearance; and the fourth, or countersunk head, is only used when a smooth surface is desirable, as over a bearing plate.

The exact sizes of heads, shapes, etc., of rivets vary in different mills.

When the size of rivet is specified the hole is always made $\frac{1}{16}$

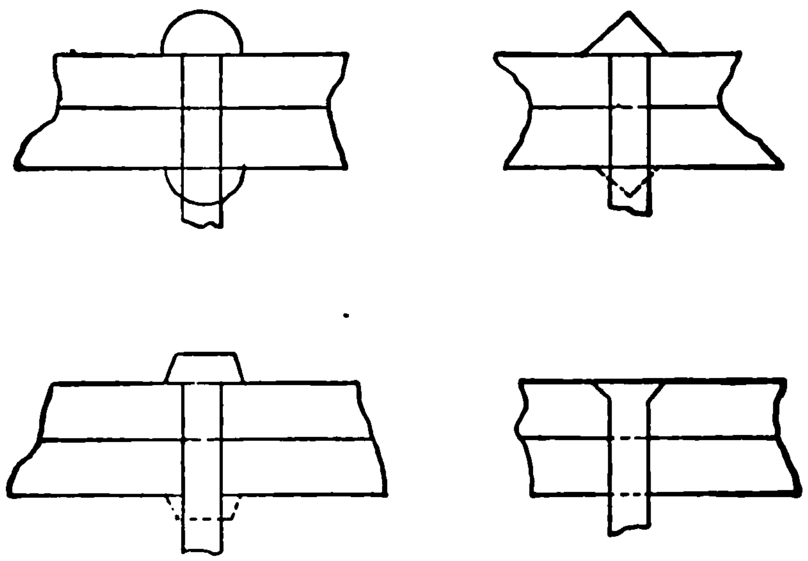

FIG. 1.

inch larger; but the rivet is generally designated by the size of the hole.

Pitch. The distance between the centres of the rivets, in the line of riveting, is called the *pitch*. This (for practical reasons) should never be less than $2\frac{1}{2}$ diameters; nor should the centre of the hole (if possible) be nearer to any edge than $1\frac{1}{2}$ diameters. In angle work, however, it is often necessary to make the distance from the edge less than the above, but in thick plates it should always be more. In drilled work the pitch might be reduced to 2 diameters. If rivet-heads are countersunk the pitch should be increased according to the amount of metal cut away, to make room for the rivet head.

Rivet-holes are generally made by punching, by a powerful steam-punch, as this is much the cheapest method. The best way to make the holes is to drill them after the pieces are bolted or clamped together.

Punching makes a ragged and irregular hole, and injures the metal about the hole, causing a loss in strength to the remaining portion of the metal of 15 per cent. in wrought-iron, and often 35 per cent. in steel.

Besides this, in punching there is liability of cracking the plate, and of not having the holes in the two plates that are to be united come exactly opposite each other.

The hardening of the metal by punching also decreases the ductility of the pieces.

The injury done by punching in steel plates may be almost entirely removed, however, by annealing, and in first-class work this should always be done.

In drilled work there is no loss, and the holes are not only accurately located, but accurately cut, and the strength of the remaining fibres is even increased from 12 to 25 per cent.

The cost of drilling, however, is very great, so that it is not likely to be employed, except in making the joints in trusses and connecting tie-bars, where the number of rivets is not great.

A medium course between punching and drilling is to punch the holes a size smaller than desired, and then drill or ream them to actual size, when partially secured together. The loss of strength by this method will be very slight.

In most cases, however, the architect will have to be satisfied with punched holes, and must, therefore, allow sufficient metal to make good any damage done, or for any inaccuracies.

In driving and heading the rivet, however, machine riveting is much better than hand riveting, as a greater pressure is used, and the metal more completely fills the hole.

In designing riveted work, whether to be hand or machine riveted, the architect should bear in mind the necessity of placing the rivets so that they can be inserted in the holes from one side and hammered from the other; and for machine work, that the machine can reach them. Thus, the minimum distance from the inside face of one leg of an angle iron to centre of nearest rivet-hole in other leg should be at least $1\frac{1}{8}$ inch for $\frac{7}{8}$-inch rivets, 1 inch for $\frac{3}{4}$-inch rivets, $\frac{7}{8}$ inch for $\frac{5}{8}$-inch rivets, $\frac{13}{16}$ inch for $\frac{1}{2}$-inch rivets; and, if possible, these distances should be increased.

Riveted joints may yield in any one of five ways:

1st. By the crushing of the plate in front of the rivets (Fig. 2).

2d. By the shearing of the rivets (Fig. 3).

3d. By the tearing of the plate between the rivet-holes (Fig. 4).

4th. By the rivet breaking through the plate (Fig. 5).

5th. By the rivet shearing out the plate in front of it.

The two latter cases are likely to occur only in the case of a single riveted lap-joint.

To design a riveted joint so that it will not break in either of these ways, it is, therefore, necessary to calculate for the shearing strength of the rivets, for the crushing strength of the plates joined, and to space the rivets far enough apart that the metal will not tear between the rivets.

The process of designing a riveted joint practically consists in first assuming the size of rivet to be used, and then calculating the number required to resist shearing and to prevent the crushing of the plates joined, and then using the larger number. They are

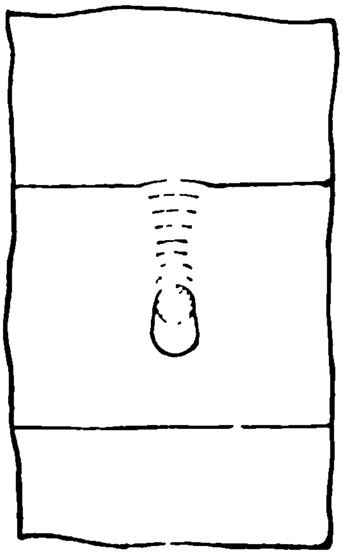

FIG. 2.

FIG. 3.

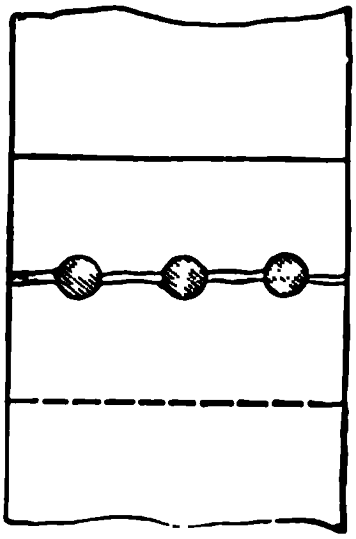

FIG. 4.

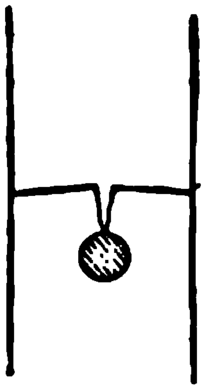

FIG. 5.

then spaced by the rule that the pitch shall not be less than 2¼ diameters, nor more than 16 times the thickness of the thinnest plate at the joint, and the distance from the centre of the rivet to end of the plate should not be less than 1½ diameters.

The following table gives the sizes of rivets to be preferred for different thicknesses of plates:

For plates from ¼ inch to $\frac{7}{16}$ inch thick, use rivet-holes ⅝ inch in diameter.

For plates from ½ inch to ⅝ inch thick, use rivet-holes ¾ inch in diameter.

For plates from $\frac{11}{16}$ inch to $\frac{13}{16}$ inch thick, use rivet-holes ⅞ inch in diameter.

For plates from ⅞ inch to 1 inch thick, use rivet-holes 1 inch in diameter.

The number of rivets required to *resist shearing* can be easily determined by dividing the total amount of strain by the number opposite the size of the rivet, in the fourth column of the following table, if the rivet is in single shear; and, if in double shear, take one-half the number of rivets.

SHEARING AND BEARING VALUE OF RIVETS.

Diameter of Rivet, in Inches.		Area of Rivet.	Single shear at 7,500 lbs. per sq. inch.	Bearing value for different thicknesses of plate at 15,000 pounds per square inch. (= Diameter of rivet × thickness of plate × 15,000 pounds.)										
Fraction.	Decimal.			$\frac{1}{4}''$	$\frac{5}{16}''$	$\frac{3}{8}''$	$\frac{7}{16}''$	$\frac{1}{2}''$	$\frac{9}{16}''$	$\frac{5}{8}''$	$\frac{11}{16}''$	$\frac{3}{4}''$	$\frac{13}{16}''$	$\frac{7}{8}''$
$\frac{3}{8}$	.375	.1104	828	1,410	......	......	......	......	......	......	......	......	......	......
$\frac{7}{16}$	.4375	.1503	1,130	1,640	2,050	......	......	......	......	......	......	......	......	......
$\frac{1}{2}$	.5	.1963	1,470	1,880	2,340	2,810	......	......	......	......	......	......	......	......
$\frac{9}{16}$	.5625	.2485	1,860	2,110	2,640	3,160	3,690	......	......	......	......	......	......	......
$\frac{5}{8}$	.625	.3068	2,300	2,340	2,930	3,520	4,100	......	......	......	......	......	......	......
$\frac{11}{16}$	.6875	.3712	2,780	2,580	3,220	3,870	4,510	5,160	......	......	......	......	......	......
$\frac{3}{4}$	.75	.4418	3,310	2,810	3,520	4,220	4,920	5,630	6,330	......	......	......	......	......
$\frac{13}{16}$	.8125	.5185	3,890	3,050	3,810	4,570	5,330	6,090	6,860	7,620	......	......	......	......
$\frac{7}{8}$	.875	.6013	4,510	3,280	4,100	4,920	5,740	6,560	7,380	8,200	......	......	......	......
$\frac{15}{16}$	.9375	.6903	5,180	3,520	4,390	5,270	6,150	7,030	7,910	8,790	9,670	......	......	......
1	1.0	.7854	5,890	3,750	4,690	5,620	6,560	7,500	8,440	9,380	10,310	11,250	......	......
$1\frac{1}{16}$	1.0625	.8866	6,650	3,980	4,980	5,980	6,970	7,970	8,960	9,960	10,960	11,950	12,950	.....
$1\frac{1}{8}$	1.125	.9940	7,460	4,220	5,270	6,330	7,380	8,440	9,490	10,550	11,600	12,660	13,710	14,770
$1\frac{3}{16}$	1.1875	1.1075	8,310	4,450	5,570	6,680	7,790	8,910	10,020	11,130	12,250	13,360	14,470	15,590

To find the number of rivets required to prevent crushing, divide the total amount of strain by the bearing value of the rivet given in the preceding table.

The heavy zigzag line in the table indicates the limit at which the bearing value exceeds single shear. All values above these lines are in excess of single shear; all values below are less than single shear.

The principal cases in which riveted joints occur in building construction are:

1. In the joints of wrought-iron trusses.
2. Splicing of tie-bars.
3. In the connecting angles of floor beams.
4. In riveted girders.

Splicing of Tie-bars.

Tie-bars may be spliced in three ways.

1st. By a lap-joint, as shown in Fig. 6.

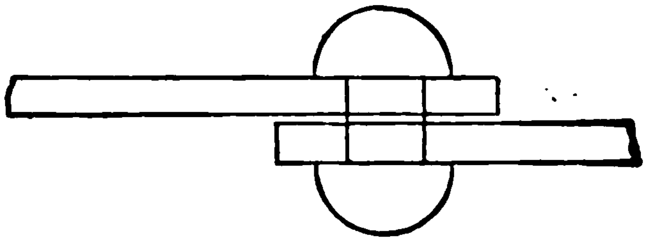

FIG. 6.

2d. By a single cover plate, as shown in Fig. 7.

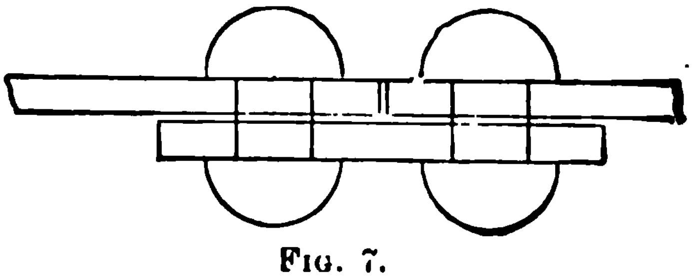

FIG. 7.

3d. By two cover plates, as in Fig. 8.

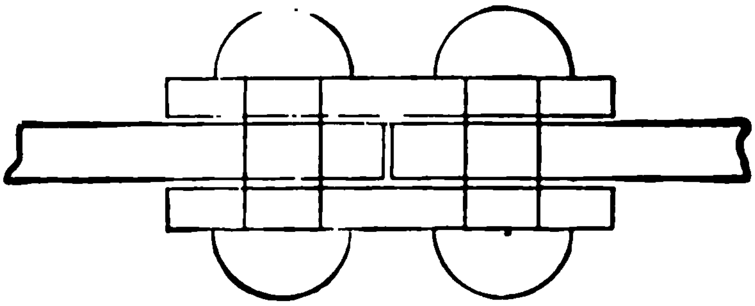

FIG. 8.

In Figs. 6 and 7 the rivets are in single shear; in Fig. 8 they are in double shear. The last method is much the best, although it is also the most expensive. The cover plates should always be the

full width of the bars connected, and $\frac{1}{16}$ inch more in thickness for the two plates, or for one single plate.

For lapped joints, which is the most common joint used, the rivets should be arranged as in Fig. 9, in which case the plates are

FIG 9.

only weakened by the width of one rivet-hole, at A. At B, two rivet-holes are lost, but the strain has been reduced by an amount equal to the value of one rivet-hole, and so on.

If the plates are narrow and thick, the rivets may be arranged as in Fig. 10 or 11.

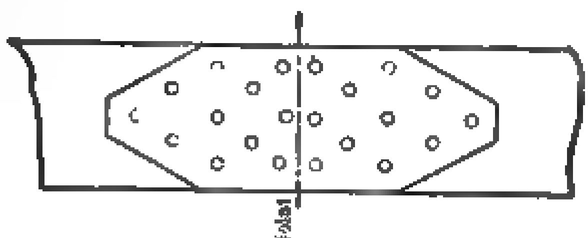

FIG. 10.

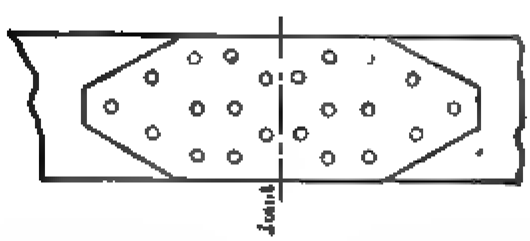

FIG. 11.

Where cover plates are used, Fig. 11 is the best arrangement to use, for then the cover plates will be weakened by only two rivet-holes (the ones nearest the joint); while in Fig. 10 the cover plates are weakened by three holes nearest the joint, and, consequently, must be made thicker.

When rivets are arranged in rows, it is called chain riveting; when rivets are arranged to come opposite the space between the preceding rivets, they are said to be staggered, as in Figs 9, 10, and 11.

In designing riveted joints care must be exercised not to weaken the plates any more than is absolutely necessary.

EXAMPLE 1.—*A 12″ × ½″ tie-bar is so long that it has to be made in two pieces with a splice; the strain on the piece is* 65,000 *pounds. How many rivets will be required?*

Ans. We will assume that the joint is to be a lapped joint, as in Fig. 9, and that we will use ¾-inch rivets.

From the table we find that the resistance of a ¾-inch rivet to single shear is 3,310 lbs. and the bearing value for a ½-inch plate 5,630 lbs. Dividing the strain, 65,000 lbs., by the smaller of these two quantities, 3,310, we find we shall require 20 rivets; but as 20 rivets will not give us the arrangement we wish, we will use 25, as in Fig. 9. The distance, P, between the centres of rivets measured on the slant should be at least 2½ diameters, or 2½ × ¾ inch = 1⅞ inches, or, we will say, 2 inches.

Beam Connections.

EXAMPLE 2.—*A* 10 *inch iron beam having a web* 5/16 *inch thick sustains a distributed load of* 12,000 *lbs. One end of the beam rests on a wall, the other is framed to a* 15-*inch I-beam girder; how many rivets will be required in the connection?*

Ans. The standard connections (see p. 368) show two 3½ × 3½ × ⅜ angles, with two ¾-inch bolts, and we will see if it is strong enough for this particular case.

The load on the beam being 12,000 lbs., only one-half will be transferred to the girder, or 6,000 lbs.; hence the two ¾-inch rivets will be required to support 6,000 lbs.

From the table the bearing value of a ¾-inch rivet on a 5/16-inch plate is 3,520 lbs., which for the two rivets will be 7,040 lbs. The rivets will be in double shear, hence will have the same strength as 4 rivets in single shear.

The value for one rivet is 3,310 lbs., or 13,240 lbs. for the 2 rivets in double shear—or more than twice as strong. The angles are thicker than the web, hence the bearing strength on them is greater than on the web. We therefore find the standard connection has sufficient strength for this particular case, with no excessive waste.

Rivets in Plate Girders.—It is quite a difficult matter to scientifically proportion the rivets in plate girders, so the common practice is to put in enough to meet both the practical and theoretical requirement

The usual practice is to use ¾-inch rivets, spaced from 4 to 6 inches apart according to the size of the girder, and not more than

3 or 4 inches apart at the ends. In very light girders having plates less than $\frac{1}{2}$ inch in thickness, $\frac{5}{8}$-inch rivets may be used.

Bending Moment in Rivets.—While pins should always be computed for resistance to cross breaking, it is not the custom to consider the bending moment in rivets; as in a well-riveted joint it is practically impossible to produce any bending of the rivet, neither do the tests on riveted joints show any signs of the rivets breaking in that way. The only person that considers the bending moment on rivets, so far as the author has been able to learn, is Mr. Louis DeCoppet Berg, who has taken up the subject of riveted joints most elaborately in Chapter IX. of his papers on "Safe Building," published in the *American Architect and Building News*, in the latter part of the year 1889.

PART III.

RULES, MEMORANDA, AND TABLES

USEFUL IN

DESIGNING, ESTIMATING, AND BUILDING.

CHIMNEYS.

[From the "Building and Engineering Times."]

The object of a chimney is to produce the draught necessary for the proper combustion of the fuel, as well as to furnish a means of discharging the noxious products of combustion into the atmosphere at such a height from the ground that they may not be considered a nuisance to people in the vicinity of the chimney.

Regarding the second of the above purposes for which chimneys are built, it need only be said, that it is of secondary importance only, and that where due attention is given to the proper methods of setting boilers, and proportionating grate areas, furnaces, and rate of combustion, the smoke nuisance is comparatively unknown, and is of no practical importance whatever.

The main points to be considered in designing chimneys are the right proportions to insure, first, a good and sufficient draught, and, second, stability.

Without entering into any demonstration of the velocity of the flow of the heated gases through the furnace and flues leading into and up the chimney, we will briefly state a few of the principles governing the dimensions of chimneys. The motive power or force which produces the draught is the action of gravity upon the difference in the specific gravities of the heated column of the gases of combustion inside the chimney, and the atmosphere at its normal temperature outside of the chimney, by which the former is forced up the flue; and the laws governing its velocity are the same as those governing the velocity of a falling body; and it can be proved that its velocity, and, consequently, the amount or volume of air drawn into the furnace, and which constitutes the draught, is in proportion to the square root of the height of the chimney. It is a common error that the force of the draught is in direct proportion to the height; so that, with two chimneys of the same area of flue, one being twice the height of the other, the higher one would produce a draught twice as strong as the other. The intensity of draught under these circumstances would be in the proportion of the square root of 1 to the square root of 2, or as 1 to 1.42. To double the draught-power of any given chimney by adding to the height, it would be necessary to build it to four times the origi-

nal height. Practically there is a limit to the height of a chimney of any given area of flue, beyond which it is found that the additional height increases the resistance due to the velocity and friction more rapidly than it increases the flow of cold air into the furnace. For chimneys not over forty-two inches in diameter the maximum admissible height is about three hundred feet.

From an investigation of the same laws we find that the velocity of the flow of cold air into the furnace is in proportion to the square root of the ratio between the density of the outside air and the difference in the densities of the outside air and the heated gases in the chimney; from which we may deduce the fact that very little increase of draught is obtained by increasing the temperature of the gases in the chimney above 550 or 600 degrees F. By raising the temperature of the flue from 600 to 1200 degrees we would increase the draught less than twenty per cent. while the waste of heat would be very considerable. Conversely, we may reduce the temperature of the flue about one-half, when the temperature is as high as six hundred degrees, by means of an economizer or otherwise, and the *reduction* of draught-force would be only about twenty per cent, as before.

It is found that the principal causes which act to impair the draught of a chimney, and which vary greatly with different types of boilers and settings, are the resistance to the passage of the air offered by the layer of fuel, bends, elbows, and changes in the dimensions of the flues, roughness of the masonry of brick flues, holes in the passages which allow the entrance of cold air, and, generally, any variation from a straight, air-tight passage of uniform size from combustion-chamber to chimney-flue; and the resistance to draught is in direct proportion to the magnitude and number of such variations.

In designing a chimney, it is, therefore, always necessary to consider the type of boiler, method of setting, arrangement of boilers and flues, location of chimney, and every thing which will be likely to in any way interfere with its efficient performance. Much, of course, depends upon the judgment and experience of the designer, and it would be impossible to give any general rule which would cover all cases. When only one boiler discharges into a chimney, for instance, the chimney requires a much larger area per pound of fuel burned than when several similar boilers discharge into a chimney of the same height; and, taking all these varying circumstances into consideration, a great deal of judgment is, in many cases, required to determine the proper dimensions.

It is a common idea that a "chimney cannot be too large;" in other words, the larger the area of the flues, the better the draught

ll be. But this is not always the case. In many cases where a imney has been built large enough to serve for future additions the boiler-power, the draught has been much improved as addi- onal boilers have been set at work. The cause of this is to be und in the increased steadiness of draught where several boilers e at work and are fired successively, as well also as in the better aintenance of the temperature of the flue; as the velocity of the ses necessarily increases with the increased amount required to discharged, and they do not have time to cool off to so great an tent as when they move more slowly.

RULES FOR PROPORTIONING CHIMNEYS.

[Published by the Babcock & Wilcox Co., of New York.]

Chimneys are required for two purposes—1st, to carry off obnox- us gases; 2d, to produce a draught, and so facilitate combustion. he first requires **size**, the second **height.**

Each pound of coal burned yields from 13 to 30 pounds of gas, e volume of which varies with the temperature.

The weight of gas carried off by a chimney in a given time pends upon three things—size of chimney, velocity of flow, and nsity of gas. But as the density decreases directly as the absolute mperature, while the velocity increases, with a given height, arly as the square root of the temperature, it follows that there a temperature at which the weight of gas delivered is a maximum. his is about 550° above the surrounding air. Temperature, how- er, makes so little difference, that at 550° above, the quantity is *ly four per cent.* greater than at 300°. Therefore, height and ea are the only elements necessary to consider in an ordinary imney.

The intensity of draught is, however, independent of the ze, and depends upon the difference in weight of the outside and side columns of air, which varies directly with the product of the ight into the difference of temperature. This is usually stated an equivalent column of water, and may vary from 0 to possibly inches.

To find the maximum draught for any given chimney, e heated column being 612° F., and the external air 62°: *Multiply e height above grate in feet by* **.0075,** *and the product is the aught power in inches of water.*

The intensity of draught required varies with the kind and ndition of the fuel, and the thickness of the fires. Wood requires e least, and fine coal or slack the most. To burn anthracite

slack to advantage, a draught of $1\frac{1}{2}$ inch of water is necessary, which can be attained by a well-proportioned chimney 175 ft. high.

A round chimney is better than square, and a straight flue better than tapering, though it may be either larger or smaller at top without detriment.

The effective **area of a chimney,** for a given power, varies inversely as the square root of the height. The actual area, in practice, should be greater, because of retardation of velocity due to friction against the walls. On the basis that this is equal to a layer of air two inches thick over the whole interior surface, and that a commercial horse-power requires the consumption of an average of 5 pounds of coal per hour, we have the following formulæ:

$$E = \frac{0.3\,H}{\sqrt{h}} = A - 0.6\sqrt{A} \qquad (1)$$

$$H = 3.33\,E\sqrt{h} \qquad (2)$$

$$S = 12\sqrt{E} + 4 \qquad (3)$$

$$D = 13.54\sqrt{E} + 4 \qquad (4)$$

$$h = \left(\frac{0.3\,H}{E}\right)^2 \qquad (5)$$

In which H = horse-power; h = height of chimney in feet; E = effective area, and A = actual area in square feet; S = side of square chimney, and D = dia. of round chimney in inches. The following table is calculated by means of these formulæ, by MR. WM. KENT:

SIZES OF CHIMNEYS WITH APPROPRIATE HORSE-POWER OF BOILERS.

Diameter in inches	Height 50 ft.	60 ft.	70 ft.	80 ft.	90 ft.	100 ft.	110 ft.	125 ft.	150 ft.	175 ft.	200 ft.		Effective area, sq. ft.	Actual area, sq. ft.
18	23	25	27										0.97	1.77
21	35	38	41										1.47	2.41
24	49	54	58	62									2.08	3.14
27	65	72	78	83									2.78	3.98
30	84	92	100	107	113								3.58	4.91
33		115	125	133	141							30	4.48	5.94
36		141	152	163	173	182						32	5.47	7.07
39			183	196	208	219						35	6.57	8.30
42			216	231	245	258	271					38	7.76	9.62
48				311	330	348	365	389				43	10.44	12.57
54					427	449	472	503	551			48	13.51	15.90
60					536	565	593	632	692	748		54	16.98	19.64
66						694	728	776	849	918	981	59	20.83	23.76
72						835	876	934	1023	1105	1181	64	25.08	28.27
78							1038	1107	1212	1310	1400	70	29.73	33.18
84							1214	1294	1418	1531	1637	75	34.76	38.48
90								1496	1639	1770	1893	80	40.19	44.18

The external diameter at the base should be one-tenth the height, unless it be supported by some other structure. The "**batter**" or **taper** of a chimney should be from $\frac{3}{16}$ to $\frac{1}{4}$ inch to the foot on each side.

Thickness of brick work: one brick (8 or 9 inches) for 25 feet from the top, increasing $\frac{1}{2}$ brick (4 or $4\frac{1}{2}$ inches) for each 25 feet from the top downward

If the inside diameter exceed 5 feet the top length should be $1\frac{1}{2}$ bricks, and if under 3 feet it may be $\frac{1}{2}$ brick for ten feet.

EXAMPLES OF LARGE CHIMNEYS.[1]

The Townsend Chimney, Port Dundas, Glasgow.—This is one of the tallest, if not *the* tallest chimney in the world. It was designed by Mr. Robert Corbett, of Glasgow, for Mr. Joseph Townsend, of the Crawford Street Chemical Works. It rests on blue clay, "solid as a rock."

The foundation consists of thirty courses of bricks on edge, the lowest course being 50 feet and the top course 32 feet in diameter. The inside lining, or cone, is of 9-inch fire-brick and 60 feet in height, built distinct from the chimney proper, with air space between and covered on top to prevent dust from falling in, but built with open work in the upper four courses, so as to allow of air passing into the chimney.

The chimney is 454 feet high above the ground level, and is built of brick, the thickness of the wall varying as follows:

1st	section,	30	feet	in height,	5	feet	7	inches thick.
2d	"	30	"	"	5	"	2	" "
3d	"	30	"	"	4	"	10	" "
4th	"	40	"	"	4	"	5	" "
5th	"	40	"	"	4	"	0	" "
6th	"	40	"	"	3	"	7	" "
7th	"	40	"	"	3	"	2	" "
8th	"	40	"	"	2	"	9	" "
9th	"	40	"	"	2	"	4	" "
10th	"	52	"	"	1	"	11	" "
11th	"	52	"	"	1	"	7	" "
12th	"	20	"	"	1	"	2	" "

Iron hoops were built in the chimney at a distance of 9 inches from the surface at the bottom and $4\frac{1}{2}$ inches at the top, and at in-

[1] The best modern work on Tall Chimney Construction is by R. M. and F. J. Bancroft, published in England, for sale by W. T. Comstock, New York.

tervals of 25 feet in height. When nearly completed the chimney was struck by a severe gale, which, together with a fault in the construction of the scaffolding, caused it to lean 7 feet and 9 inches, and the chimney was brought to a perpendicular by means of twelve cuttings with saws on the opposite side of the inclination.

The chimney was completed October 6, 1859. It has been several times struck by lightning, but not seriously damaged. It is protected by two ¾-inch copper lightning-rods.

St. Rollox Chemical Works Chimney, Glasgow: Dimensions.—Height from foundation to top, 455 feet 6 inches.

Height from ground surface to top, 436 feet 6 inches.

Outside diameter at foundation, 50 feet.

Outside diameter at ground surface, 40 feet.

Outside diameter at top, 13 feet 6 inches.

Height of inner cone from foundation to top, 263 feet.

Height of inner cone from ground surface to top, 243 feet.

Inner cone, inside diameter at foundation, 12 feet.

Inner cone, inside diameter at top, 13 feet 6 inches.

The outline of the chimney is similar to that of the Eddystone Lighthouse.

Chimney-stack of Messrs. Dobson & Barlow, Kay Street Machine Works, Bolton, Lancashire, Eng.—Total height from ground level, 367 feet 6 inches.

Octagonal in plan, 14 feet on each side, or 112 feet in girth at the bottom.

Thickness of brickwork at the bottom, 8 feet.

Thickness of brickwork at the top, 1 foot 6 inches.

Size at top, 5 feet 6 inches, each side: or 44 feet in girth.

Eight hundred thousand brick and 120 tons of stone-work were consumed in the building. The top, with cornices and mouldings, required 30 tons of stone and cement. (This is the highest chimney-stack in England.)

Chimney-stack at the West Cumberland Hematite Iron Works. Designed by Professor J. Macquorn Rankine, and considered as a model chimney.

Duty. The duty of this chimney is to carry off the gaseous products of combustion from four blast furnaces and from various stoves and boilers. The total amount of fuel consumed is estimated at about 10[illegible] tons per hour, when all the furnaces are at work.

The actual temperature inside the chimney when doing about three-fourths of its full duty is 190° F., and the pressure of the draught is 1 inches of water.

Figure and Dimensions.—Above ground the chimney is a frustum of a cone, with a straight batter. Underground there is a plinth or basement, octagonal outside at the ground line, and square at the bottom; cylindrical inside, and pierced with four circular openings for flues.

Height of chimney above the ground, 250 feet.

Depth of foundation below the ground, 17 feet.

Total height from foundation to top, 267 feet.

Inside diameter at top of cone, 13 feet.

Inside diameter, two feet above bottom of cone, 21 feet 10 inches.

Inside diameter in basement, 18 feet 10 inches.

Inside diameter of archway for flues, 7 feet 6 inches.

Outside diameter at top of cone, 15 feet 3 inches.

Outside diameter 2 feet above bottom of cone, 25 feet 7 inches.

Outside dimensions of square basement, 30 feet × 30 feet.

Size of foundation course, 31 feet 6 inches × 31 feet 6 inches.

Size of concrete foundations, 34 feet 6 inches × 34 feet 6 inches, and 3 feet thick.

Thickness of Brickwork.—First two feet above foundation stepping from 4 bricks to 2½ bricks; next 88 feet, 2½ bricks; next 80 feet, 2 bricks; remaining 80 feet, 1½ bricks.

The pressure on the ground below the concrete is 1.6 tons on the square foot.

Fire-brick Lining.—The thickness of brickwork given above included the fire-brick lining, which was one brick in thickness in the first 90 feet, and ½ brick the remaining height, the fire-brick being bonded in with the common brick, but being laid in fire-clay. This method of construction was considered better than that of the inner cone.

Strips of No. 15 hoop iron, tarred and sanded, were laid in the bed-joints of the cone at intervals of 4 feet in height, with their ends turned down in the side-joints. The length of the iron was twice the circumference of the chimney.

Cap and Lightning Conductor.—On the top of the chimney is a pitch-coated cast-iron curb, one inch thick, coming down three inches on the outside and inside. The lightning conductor is a copper wire rope three-fourths inch in diameter. It terminates in a covered drain, in which there is always a sufficient run of water.

"Jumbo" Chimney of the Merrimack Manufacturing Company, Lowell, Mass.—This chimney was built in 1882. It is a round chimney: height from the surface of the

ground, 282.75 feet; diameter of base, 28 feet; diameter of the narrowest part near the top, 15 feet; diameter of flue, 12 feet; the amount of staging used was 28,000 feet; the number of brick used, 1,050,000. The chimney is surmounted by a cast-iron cap of over nine tons weight, its largest diameter being 21 feet. It is protected from lightning by a three-fourths inch cable conductor with two tips. The chimney was built to accommodate 16 nests of upright Corliss boilers of 300 H. P. per nest, and its sole use is to furnish the necessary draught and convey away the smoke from these boilers. The chimney was planned and engineered by J. T. Baker, C.E., at that time for the Merrimack Company. A full description of this chimney, with plans and elevation, was published in the *Transactions of the American Society of Civil Engineers* for April, 1885, No. CCCI.

The Pacific Mills Chimney at Lawrence, Mass.—This chimney was built by Mr. Hiram F. Mills, C.E., in 1873, and consists of an outside octagonal shell 222 feet high above the ground, with a distinct interior core 8 feet 6 inches in diameter inside, extending one foot above the top of the outer shell, and 11 feet below the ground. The chimney is founded 19 feet below the ground, upon coarse sand, the foundation being 35 feet square, enclosed by pine sheet-piling. The base is concrete, 1 foot thick, then rubble masonry of large pieces of granite in cement, this stone-work being 7 feet high. Upon the stone-work is placed the brick chimney, the outer shaft being at the base 20 feet wide, and at the top, under the projecting cornice, 11 feet 6 inches wide. This brickwork is 28 inches in thickness at the base; at 12 feet in height it becomes 24 inches, which continues 18 feet; then 20 inches for 20 feet; then 16 inches for 40 feet; then 12 inches for 60 feet; then 8 inches to the top. The top of the chimney is of cast iron plates $\frac{3}{4}$ inch thick. The horizontal flue entering the chimney is 7 feet 6 inches square. The inside vertical flue of the chimney is a cylinder 8 feet 6 inches in inside diameter, and 234 feet high, with walls 20 inches thick for 20 feet, 16 inches thick for 17 feet, 12 inches thick for 52 feet, and 8 inches thick for 145 feet. The foundations were laid in mortar of Rosendale cement and sand, the outer shell in mortar of Rosendale cement, lime, and sand, and the flue walls in mortar of lime and sand.

During the winter of 1873, the flue being 90 feet above the ground, the boilers, having 152 square feet of grate surface, were connected with the chimney with satisfactory results. Between June and September, 1874 the chimney was finished. The approximate weight of the chimney is 2,250 long tons, the number of bricks

being about 550,000. The chimney is opposite the middle of a line of 28 boilers, and 210 feet distant from them. It was designed to serve for boilers having 700 square feet of grate surface, burning about 13 tons of anthracite coal per hour.

The chimney was struck by lightning in June, 1880, after which date a lightning-rod was put up, which consists of a seamless copper tube $\frac{5}{16}$" thick, 1 inch inside diameter, at the top of which are 7 points radiating from a ball 4 inches in diameter, the top of the central point being $8\frac{1}{2}$ feet above the iron cap. The rod is attached to the chimney by brass castings, and is connected at the bottom to a 4-inch drain-pipe extending 60 feet to a canal.

Chimney near Freiberg, Saxony.—Supposed to be the highest in the world (1891).

It is 460 feet high, 33 feet in diameter at its base, and 16 feet at the top, its inner diameter being 8 feet. It is built throughout of massive claystone with a facing of markstone at its base.

Wrought-iron Chimneys.[1]—"Wrought-iron shafts have found great favor in America and Russia, but in England and the Continent generally, as far as we have been able to ascertain, they are an exception. In addition to the wrought-iron shafts detailed in this paper we have been informed of the following: Messrs. Witherow & Gordon, of Pittsburgh, Penn., U. S. A., have, since 1876, built upward of thirty wrought-iron shafts, varying in height from 100 feet to 190 feet, and from 5 feet to 9 feet in diameter. The firm write us that these shafts answer admirably the purpose for which they were built. Mr. L. S. Bent, Superintendent of the Pennsylvania Steel Company, Steelton, Penn., U. S. A., states that his company have the following eight wrought-iron shafts in use, and have found them both durable and economical:

No. 1, 170	feet high,	6	feet	6 inches	diameter,	built 1881
No. 1, 165	"	6	"	"	"	" 1877
No. 1, 135	"	7	"		"	" 1880
No. 1, 112	"	6	"		"	" 1881
No. 4, 110	"	7	"		"	1869, '74, 5–6

"They are lined for 30 feet with 9-inch fire-brick, and the remainder of height with 4-inch red brick. The Ravensdale Iron Works chimney-shaft, Tunstall (Messrs. Robert Heath & Sons), is a circular wrought-iron shaft not spread at its base. Its height from ground-line to top is 75 feet; outside measurement at ground sur-

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[1] R. M. & F. J. Bancroft, Tall Chimney Construction.

FLOW OF GAS IN PIPES.

[From Haswell's "Engineers' and Mechanics' Pocket-Book."]

The flow of gas is determined by the same rules as those governing the flow of water. The pressure applied is indicated and estimated in inches of water.

DIAMETER AND LENGTH OF GAS-PIPES TO TRANSMIT GIVEN VOLUMES OF GAS TO BRANCH PIPES.

[Dr. Ure.]

Volume per hour, in cu. ft.	Diameter, in ins.	Length, in feet.	Volume per hour, in cu. ft.	Diameter, in ins.	Length, in feet.
50	0.40	100	2,000	5.32	2,000
250	1.00	200	2,000	6.33	4,000
500	1.97	600	2,000	7.00	6,000
700	2.65	1,000	6,000	7.75	1,000
1,000	3.16	1,000	6,000	9.21	2,000
1,500	3.87	1,000	8,000	8.95	1,000

The volumes of gases of like specific gravities discharged in equal times by a horizontal pipe under the same pressure, and for different lengths, are inversely as the square roots of the lengths.

The loss of volume of discharge by friction, in a pipe six inches in diameter and one mile in length, is estimated at ninety-five per cent.

Gas Memoranda.

In distilling fifty-six pounds of coal, the volume of gas produced in cubic feet, when the distillation was effected in three hours, was 41.3 ; in seven hours, 37.5 ; in twenty hours, 33.5 ; and in twenty-five hours, 31.7.

A retort produces about six hundred cubic feet of gas in five hours, with a charge of about one and a half hundred-weight of coal, or 2,800 cubic feet in twenty-four hours.

A cubic foot of good gas, from a jet one-thirty-third of an inch in diameter and a flame of four inches, will burn sixty-five minutes.

Internal lights require four cubic feet, and external lights about five cubic feet, per hour. When large or Argand burners are used, from six to ten cubic feet will be required.

fastened to the floor timbers near their tops. The pipe should be securely fastened to the support to prevent lateral movement. The drop-pipe must be perfectly plumb, and pass through a guide fastened near the bottom of the timbers, which will keep them in position despite the assaults of lathers, masons, and others. In the absence of express directions to the contrary, outlets for brackets should generally be four feet and six inches high from the floor, excepting that it is usual to put them six feet in halls, and five feet in bath-rooms. The upright pipes should be plumb, so that the nipples that project through the walls will be level. The nipples should project not more than three-quarters of an inch from the face of the plastering. Laths and plaster together are usually three-fourths of an inch thick; hence, the nipples should project one and one-half inches from the face of the studding. Drop centre pipes should project one and one-half inches below the furring, or timbers if there be no furring, where it is known that there will be no stucco or centre-pieces used. Where centre-pieces are to be used, or where there is a doubt whether they will be or not, then the drop-pipes should be left about a foot below the furring. All pipes being properly fastened, the drop-pipe can be safely taken out and cut to the right length when gas-fixtures are put on. Gas pipes should never be placed on the bottoms of floor timbers that are to be lathed and plastered, because they are inaccessible in the contingency of leakage, or when alterations are desired, and gas-fixtures are insecure. The whole system of piping should be proved to be air and gas tight under a pressure of air that will raise a column of mercury six inches high in a glass tube. The pipes are either tight or they leak. There is no middle ground. If they are tight the mercury will not fall a particle. A piece of paper should be pasted on the glass tube, even with the mercury, to mark its height while the pressure is on. The system of piping should remain under test for at least a half-hour. It should be the duty of the person in charge of the construction of the building to thoroughly inspect the system of gas-fitting; surely as much so as to inspect any other part of the building. He should know from personal observation that these specifications are complied with. After being satisfied that the mercury does not fall he should cause caps on the outlets to be loosened in different parts of the building, first loosening one to let some air escape, at the same time observing if the mercury falls, then tighten it and repeat the operation at other points. This plan will prove whether the pipes are free from obstruction or not. When he is satisfied that the whole work is properly and perfectly executed, he should give the

workmen a certificate to that effect, and no job of gas-fitting should be considered complete until such certificate is issued. The following scale of sizes of pipes and number of burners to be supplied therefrom is found by experience to be best adapted for securing a good flow of common city gas, and it is very important that it be rigidly observed when machine or air gas is to be used. Do not confound fixture outlets with burners. In establishing the sizes of pipe in a building, count the number of burners that there will be on each outlet, and have the pipes of a size to correspond therewith.

Greatest number of feet to be run.	Size of pipe.	Greatest number of bufners to be supplied.
20 feet.	3/8 inch.	2
30 "	1/2 "	4
50 "	3/4 "	15
70 "	1 "	25
100 "	1¼ "	40
150 "	1½ "	70
200 "	2 "	140
300 "	2½ "	225
400 "	3 "	300
500 "	4 "	500

STAIRS.

Wooden stairs are generally built with two-inch plank **stringers** notched out on the upper side to form the steps, and covered with pieces of boards, whose length is equal to the width of the stairs. The horizontal boards upon which the feet are placed are called the **treads;** and the vertical boards, the **risers.** In first-class work, the treads should be an inch and a quarter thick, and the risers seven-eighths ot an inch thick, and both should be of some hard wood. The stringers should not be placed over sixteen inches apart from centres, and twelve inches is better.

The treads generally project an inch and a half beyond the face of the risers, forming a **nosing.**

A good rule for the proportion of risers and treads is that the sum of the rise and tread shall equal seventeen inches and a half. Thus, if the rise is six inches, the tread should be eleven inches and a half (plus the width of the nosing); or, if the rise is eight inches, the tread should be but nine inches and a half.

The rise is always measured from top to top of treads; and the tread, from face to face of risers. The following table shows at a

glance how many risers or treads there will be in any given distance.

EXAMPLE. — In a certain building the height from the top of the first floor to the top of the second is 18 feet. How many risers will be required, and what will they be?

Ans. Find in the table the heights coming nearest to 18 feet, and then notice the height and number of risers necessary to attain this height. Thus, in the column headed $7\frac{1}{4}$ inches, at the bottom we find 18 feet $1\frac{1}{2}$ inches, showing that 30 risers $7\frac{1}{4}$ inches each will give 18 feet $1\frac{1}{2}$ inches. If we used a rise of $7\frac{1}{2}$ inches, 29 risers would also give us 18 feet $1\frac{1}{2}$ inches. Hence we shall need either 29 or 30 risers, according as we wish our rise $7\frac{1}{4}$ or $7\frac{1}{2}$ inches. If we use a rise of $7\frac{3}{4}$ inches, we shall only require 28 risers. The number of treads in a given distance can be found in the same way.

Rise x run not less than 70, not more 75
Rise + run 17 18
2 Rises + 1 run 23 25

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number runs = 1 - total number rises
i.e. height = 10' = 112" = 15 rise $7\frac{1}{2}$
if runs = $9\frac{1}{2}$; total length = $14 \times 9\frac{1}{2}$ = 11'-1"

SEATING-SPACE IN THEATRES.

[From London "Building Times."]

The question of seating is one upon which a manager and the public are apt to differ.

The requirements of the Metropolitan Board of Works with respect to seating are, that "the area to be assigned to each person shall not be less than one foot eight inches by one foot six inches, in the gallery, nor less than two feet four inches by one foot eight inches, in the other parts of the house, room, or other place of public resort." These conditions it is perhaps hardly necessary to say are not complied with in any theatre under the jurisdiction of the Board.

Until theatres are licensed to hold a certain number, or other legal restrictions enforced, an architect, in calculating the seating-capacity for the cheaper parts of his theatre, must be guided by past experience. In the upper circle, pit, and gallery, where the seats are not divided off, the audience will pack itself in an astonishing manner, when a calculation is made of the space in inches occupied by each person.

From average calculations made in London theatres, the width of seat required in the unnumbered parts of a theatre is as follows: upper circle, eighteen inches; pit, sixteen inches; amphitheatre, sixteen inches; gallery, fourteen inches. It is not intended to advocate a minimum space for the seats: on the contrary, there cannot be a doubt but that, if the minimum of eighteen inches were strictly enforced, it would be a most desirable innovation.

The several divisions of the auditorium are provided with more or less luxuriant seats according to the price paid for admission.

The stalls are usually fitted with arm-chairs, or *fauteuils*. The width of seat, and the space allowed between each row, vary considerably, according to the degree of comfort and convenience. In any case, the space allotted to each seat in the stalls is greater than that given in any other part of a theatre. The width of the seats adopted varies from twenty inches to twenty-four inches; and the distance from back to back, from three feet to five feet. The stall-seats should be the very embodiment of an easy arm-chair. A very frequent fault results from the seat being too high, and the back not sufficiently inclined. It should not be forgotten that the occupants of the stalls have to look up towards the stage. They should be able to recline easily in the chair at an angle suited to the line of vision. To sit in some stalls is to insure a stiff neck. The discomfort of stall-seats may arise from two causes, which the architect should endeavor to avoid. Firstly, the floor of the stalls

should not be sunk too low. It should never be more than four feet below the highest point of the stage-floor. Secondly, the seat should not be too high, and the back sufficiently inclined for the occupant to accommodate himself to the angle of vision. As instances of comfortable stall-chairs, the following dimensions are those of seats in two representative theatres. Width, twenty-one inches; depth, sixteen inches; height of seat from floor, sixteen inches; height from floor to top of back rail, two feet ten inches; distance from back to back, three feet ten inches. In the other case the seats are continuous, and "tip up." Width from centre to centre of arms, twenty-three inches; depth, twenty-four inches; height from floor, sixteen inches; inclination of back, 115 degrees; and the distance from back to back, three feet.

Dress-Circle. — The seats in this part are similar to those in the stalls; but the inclination of the backs should be slightly less, unless the circle is low, and not much in height above the stage level. It is also advantageous to make the seat one or two inches higher than the stall-chairs. In the theatre previously alluded to, the dress-circle seats are twenty inches wide, eighteen inches deep, eighteen inches high, and inclination of back 115 degrees. The width of the steps upon which the seats are fixed ranges from three feet to three feet six inches.

Upper Circle. — The steps in this part may be reduced to two feet six inches. This reduction in width is imperative at each level; otherwise the height of the steppings would be inconvenient. The seats should be divided by arm-rests, and have back rails. They should be eighteen inches wide, fifteen inches deep, eighteen inches high, and about 100 degrees inclination of the backs.

SPACES OCCUPIED BY SCHOOL-SEATS.

SIZES OF CHAIRS AND DESKS FOR SCHOOLS AND ACADEMIES.

Age of scholar.	Height of chair.	Height of desk next scholar.	Space occupied by desk and chair (back to back of desk).
[illegible] years.	[illegible] inches.	29½ inches.	2 feet 9 inches.
[illegible] "	15½ "	28 "	2 " 9 "
[illegible] "	15[illegible] "	27½ "	2 " 8 "
[illegible] "	14[illegible] "	26½ "	2 " 7 "
[illegible] "	1[illegible] "	2[illegible] "	2 " 5 "
[illegible] "	12[illegible] "	24 "	2 " 4 "
[illegible] "	11[illegible] "	22½ "	2 " 3 "
[illegible] "	10[illegible] "	21 "	2 " 2 "
[illegible] "	9[illegible] "	19 "	2 " 0 "

Desks for two scholars are three feet ten inches long, and for a single scholar two feet long.

SYMBOLS FOR THE APOSTLES AND SAINTS.

From the constant occurrence of symbols in the edifices of the middle ages and many of the cathedrals of the present day, the following list of symbols, as commonly attached to the apostles and saints, may be found useful: —

Holy Apostles.

St. Peter. — Bears a key, or two keys with different wards.
St. Andrew. — Leans on a cross so called from him; called by heralds the saltire.
St. John the Evangelist. — With a chalice, in which is a winged serpent. When this symbol is used, the eagle, another symbol of him, is never given.
St. Bartholomew. — With a flaying knife.
St. James the Less. — A fuller's staff, bearing a small square banner.
St. James the Greater. — A pilgrim's staff, hat, and escalop-shell.
St. Thomas. — An arrow, or with a long staff.
St. Simon. — A long saw.
St. Jude. — A club.
St. Matthias. — A hatchet.
St. Philip. — Leans on a spear, or has a long cross in the shape of a T.
St. Matthew. — A knife, or dagger.
St. Mark. — A winged lion.
St. Luke. — A bull.
St. John. — An eagle.
St. Paul. — An elevated sword, or two swords in saltire.
St. John the Baptist. — An Agnus Dei.
St. Stephen. — With stones in his lap.

Saints.

St. Agnes. — A lamb at her feet.
St. Cecilia. — With an organ.
St. Clement. — With an anchor.
St. David. — Preaching on a hill.
St. Denis. — With his head in his hands.
St. George. — With the dragon.
St. Nicholas. — With three naked children in a tub, in the end whereof rests his pastoral staff.
St. Vincent. — On the rack.

THE LARGEST RINGING BELLS IN THE WORLD.*

Names and Location of Bells.	Date cast.	Actual vibration.	Key-note.	Diameter. Inches.	Sound-bow. Inches.	Sound-bow. Stroke.	Weight. Pounds.
Moscow, Tzar Kolokol	1733	74	D	272	23.	0.84	443,772
Burmah, Mengoon	....	94	F♯	203?	16.?	0.80	201,600
Moscow, St. Ivans	1819	105	G♯	185	14.75	0.80	127,350
Pekin, Great Bell	....	....	....	156	....	......	120,000
Burmah, Maha Ganda	....	125	B	155	12.5	0.80	95,000
Nishni Novgorod	....	125	B	151	12.	0.80	69,664
Moscow, Church Redeemer	1879	141	C♯	136.3?	10.6	0.80	60,736
Nankin, China	....	....	....	112	......	....	45,000
London, St. Paul's	1881	157	E♭	114.25	8.75	0.76	42,000
Olmutz, Bohemia	....	157	E♭	121	9.125	0.75	40,320
Vienna, Austria	1711	157	E♭	118	9.5	0.80	40,200
Westminster, London	1856	166	E	113.5	9.375	0.83	35,620
Erfurt, Saxony	1487	176	F	103.6	9.75	0.75	30,800
Notre Dame, Paris	1680	166	E	103	7.5	0.73	28,670
Montreal, Canada	1847	176	F	103	7.8	0.76	28,560
York, England	1847	187	F♯	100	8.	0.80	24,080
St. Peter, Rome	1786	187	F♯	97.25	7.5	0.77	18,000
Great Tom, Oxford	1680	210	G♯	81.	6.125	0.73	17,024
Cologne, Germany	1477	198	G	95.	7.2	0.76	16,016
Brussels, Belgium	....	210	G♯	95.81	7.75	0.71	15,848
State house, Philadelphia	18[illegible]5	198	G	88.	6.375	0.73	13,000
Lincoln, England	1834	210	G♯	82.85	6.	0.73	12,0[illegible]
St. Paul's, London	1716	222	A	81.	6.08	0.75	11,500
Exeter, England	1675	210	G♯	76.	5.	0.66	10,080
Old Lincoln, England	1610	249	B	75.5	5.94	0.78	9,856
Westminster, London	1857	249	B	72.	5.75	0.79	8,960

WEIGHT OF OTHER LARGE BELLS.

City Hall, New York, 22,300 lbs.

Fire Alarm, 33d Street, New York, 21,612 lbs.

John W. Nystrom, in the Journal of the Franklin Institute.

Linden, Germany, 10,854 lbs.
Lewiston, Me., 10,288 lbs.
Rouen, France, 40,000 lbs.

DIMENSIONS OF THE PRINCIPAL DOMES.

LIST OF THE PRINCIPAL DOMES IN THE WORLD.

Their diameter, and height from the ground.

[Gwilt's Encyclopædia.]

Place.	Diameter, feet.	Height, feet.
Pantheon, at Rome	142	143
Duomo, or Sta. Maria del Fiore, at Florence,	139	310
St. Peter's, at Rome	139	330
Sta. Sophia, at Constantinople	115	201
Baths of Caracalla (ancient)	112	116
St. Paul's, London	112	215
Mosque of Achmet	92	120
Chapel of the Medici	91	199
Baptistery, at Florence	86	110
Church of the Invalids, at Paris	80	173
Minerva Medica, at Rome	78	97
Madonna della Salute, Venice	70	133
St. Généviève, at Paris (Pantheon)	67	190
Duomo, at Sienna	57	148
Duomo, at Milan	57	254
St. Vitali's, at Ravenna	55	94
Val de Grace, at Paris	55	133
San Marco, Venice	44	
United-States Capitol, Washington	124¾	

DIMENSIONS OF SOME LARGE CLOCK FACES.

Tower Clock, Depot of the Central Railroad of New Jersey, at Communipaw.—Diameter of single dial, 14 feet 3 inches; minute hand is 7 feet long, weighs 40 pounds; hour hand is 5 feet long, weighs 28 pounds.

The motive power is furnished by a weight of 700 pounds, hung from a ⅜-inch steel cable.

Four-dial Clock, New York Produce Exchange.—Diameter of each dial, 12 feet 6 inches.

Four-dial Clock, Chronicle Tower, San Francisco.—Diameter of each dial, 16 feet 6 inches; length of minute hands, 8 feet; length of hour hands, 5 feet 6 inches.

The mechanism of the clock is 6 feet and 1 inch high, and weighs 8,000 pounds.

HEIGHT OF SOME OF THE TALLEST BUILDINGS IN THE UNITED STATES.

BUILDINGS IN NEW YORK CITY.

Height from sidewalk :

Washington Building—E. H. Kendall, architect—to top of roof, 168 feet.

World Building—Geo. B. Post, architect—to top of roof, 194 feet ; to top of tower, 309 feet.

Times Building—Geo. B. Post, architect—to top of roof, 183 feet.

Equitable Building—Geo. B. Post, architect—to top of roof, 142 feet ; to top of tower, 170 feet.

Union Trust Building—Geo B. Post, architect—to top of roof, 148 feet ; to top of tower, 194 feet.

Madison Square Garden—McKim, Mead & White, architects—to top of tower, 300 feet.

BOSTON.

Ames Building—Shepley, Rutan & Coolidge, architects—to top of cornice, 186 feet.

Chamber of Commerce—Shepley, Rutan & Coolidge, architects—to top of main cornice, 93 feet ; to top of tower, 172 feet 6 inches.

PHILADELPHIA.

New City Hall—John McArthur, Jr., architect—to top of tower, 537 feet 4 inches.

CHICAGO.

Masonic Temple—Burnham & Root, architects—to top of cornice, 234 feet 2 inches ; to roof line, 278 feet 10 inches ; to top of skylight, 303 feet.

Woman's Temple—Burnham & Root, architects—to top of cornice, 141 feet ; to ridge, 198 feet.

Auditorium Building—Adler & Sullivan, architects—to cornice, 145 feet ; to top of lantern, 270 feet.

Allegheny County Court House, Pittsburgh—H. H. Richardson and Shepley, Rutan & Coolidge, architects—to ridge, 128 feet ; to top of finial, 319 feet.

Masonic Building, Pittsburgh—Shepley, Rutan & Coolidge, architects—to top of roof, 129 feet 6 inches.

State Capitol, Hartford, Conn.—R. M. Upjohn, architect—to top of roof, 99 feet ; to top of figure on dome, 256 feet.

HEIGHTS OF COLUMNS, TOWERS, DOMES, SPIRES, ETC.

COLUMNS.

Name.	Place.	Feet.
Alexander	St. Petersburg	175
Bunker Hill	Charlestown, Mass.	$221\frac{1}{6}$
Chimney (St. Rollox)	Glasgow	$455\frac{1}{2}$
Chimney (Musprat's)	Liverpool	406
City	London	202
July	Paris	157
Napoleon	Paris	132
Nelson's	Dublin	134
Nelson's	London	171
Place Vendôme	Paris	136
Pompey's Pillar	Egypt	114
Trajan	Rome	145
Washington	Washington	555
York	London	138

TOWERS AND DOMES.

Name.	Place.	Feet.
Tower	Babel	680
Tower	Baalbec	500
Capitol	Washington	$287\frac{1}{2}$
Cathedral	Antwerp	476
Cathedral	Cologne	501
Cathedral	Cremona	392
Cathedral	Escurial	200
Cathedral	Florence	384
Cathedral	Milan	438
Cathedral	St. Petersburg	363
Leaning Tower	Pisa	188
Porcelain	China	200
St. Paul's	London	366
Strasbourg	Venice	328
St. Mark's	City Hall, Philadel-	
Utrecht	phia	$537\frac{1}{3}$

HEIGHT OF SPIRES.

Name.	Place.	Feet.
Cathedral, new	New York	325
Grace Church	New York	216
Cathedral	Salisbury	450
St. John's	New York	210
St. Paul's	New York	200
St. Mary's	Lübeck	404
St. Peter's	Rome	391
St. Stephen's	Vienna	465
Trinity Church	New York	286
Balustrade of Notre Dame	Paris	216
Hôtel des Invalides	Paris	344
Pyramid of Cheops	Egypt	520
Pyramid of Sakara	Egypt	356
St. Peter's	Rome	518

CAPACITY OF SEVERAL CHURCHES, THEATRES, AND OPERA-HOUSES,

Estimating a person to occupy an area of 19.7 inches square.

CHURCHES.

St. Peter's	54,000	Notre Dame, Paris	21,000
Milan Cathedral	37,000	Pisa Cathedral	13,000
St. Paul's, Rome	32,000	St. Stephen, Vienna	12,400
St. Paul's, London	25,600	St. Dominic's, Bologna	12,000
St. Petronio, Bologna	24,400	St. Peter's, Bologna	11,400
Florence Cathedral	24,300	Cathedral of Sienna	11,000
Antwerp Cathedral	24,000	St. Mark's, Venice	7,000
St. Sophia's, Constantinople	23,000	Spurgeon's Tabernacle	7,000
St. John Lateran	22,900		

THEATRES AND OPERA-HOUSES.

Carlo Felice, Genoa	2560	Opera House, Berlin	1636
Opera House, Munich	2370	New-York Academy	1526
Alexander, St. Petersburg	2332	Philadelphia Academy	3124
San Carlos, Naples	2240	Boston Theatre, Boston	
Imperial, St. Petersburg	2160	Madison Square Theatre, New York	
La Scala, Milan	2113	Metropolitan Opera-House, New York	3500
Academy of Paris	2092	Globe Theatre, Boston	
Drury Lane, London	1948		
Covent Garden, London	1684		

DIMENSIONS OF THEATRES AND OPERA-HOUSES.

The following are the dimensions of some of the prominent theatres in this country and in Europe: —

a These dimensions include the distance between the footlights and curtain.
b Total depth of auditorium.
c Total width of auditorium.

PRINCIPAL DIMENSIONS OF THE ENGLISH CATHEDRALS.

[Gwilt.]

Cathedral.	Total internal length, in ft.	Naves and Aisles.			Choirs.			Transepts.	Spires and Towers.	
		Length, in ft.	Breadth, in ft.	Height, in ft.	Length, in ft.	Breadth, in ft.	Height, in ft.	Breadth, in ft.		Height, in ft.
Winchester	545	247	86	78	138	–	73	186		
Ely	517	327	73	70	101	73	70	178	Tower	210
Canterbury	514	214	70	80	150	74	80	154	Tower	235
Old St. Paul's	500	335	91	102	165	42	88	248	Spire	534
York	498	264	109	99	131	–	99	222	Tower	234
Lincoln	498	–	83	83	–	–	–	227	Tower	260
Westminster	489	130	96	101	152	–	151	189		
Peterborough	480	231	78	78	138	–	78	203	Louvre	150
Salisbury	452	246	76	84	140	–	84	210	Spire	387
Durham	420	–	–	–	117	33	71	176	Tower	214
Gloucester	420	174	84	67	140	–	86	144	Tower	225
Lichfield	411	213	67	–	110	–	67	–	Spire, 258 W.,	183
Norwich	411	230	71	–	165	–	–	191	Spire	317
Worcester	410	212	78	–	126	–	74	130	Tower	196
Chichester	401	205	91	61	100	–	–	131	Spire	267
Exeter	390	173	74	69	131	–	69	140	Tower	130
Wells	371	191	67	67	106	–	67	135	Tower	160
Hereford (ancient)	370	144	68	68	105	–	64	140		
Chester	348	–	73	73	–	–	–	–	Tower	127
Rochester	306	150	65	–	156	–	–	122	Spire	156
Carlisle	213	–	71	71	137	71	–	–		
Bath	210	136	72	78	–	–	–	126	Tower	162
Bristol	175	100	75	73	100	–	–	128	Tower	127
Oxford	154	74	54	41	80	–	37½	102	Spire	184

DIMENSIONS OF THE VARIOUS OBELISKS EXISTING AT THE PRESENT TIME.

[Gwilt's Encyclopædia.]

SITUATION.	Height, in English feet.	Thickness, in English feet.	
		At top.	Below.
Two large obelisks mentioned by Diodorus Siculus	158.2	7.9	11.8
Two obelisks of Nuncoreus, son of Sesostris, according to Herodotus, Diodorus Siculus, and Pliny	121.8	6.6	10.5
Obelisk of Rhameses, removed to Rome by Constantius	118.4	6.2	10.2
Two obelisks, attributed by Pliny to Smerres and Eraphius	106.0	5.9	9.8
Obelisks of Nectanabis, erected near the Tomb of Arsinöe by Ptolemy Philadelphus	105.5	5.3	9.2
Obelisk of Constantius, restored and erected in front of S. Giovanni Laterano, at Rome . . .	105.5	6.2	9.6
Part of one of the obelisks of the son of Sesostris, in the centre of the piazza in front of St. Peter's,	82.4	5.8	9.4
Two at Luxor	79.1	5.3	8.0
Obelisk of Augustus, from the Circus			
Maximus, now in the Piazza del Popola at Rome .	78.2	4.5	7.4
Two in the ruins at Thebes, still remaining . . .	72.8	5.0	7.5
Obelisk of Augustus, raised by Pius VI. in the Piazza di Monte Citorio	71.9	4.9	7.9
Two obelisks: one at Alexandria, vulgarly called Cleopatra's Needle, and the other at Heliopolis .	67.1	5.1	8.1
Obelisk by Pliny, attributed to Sothis	63.3	4.5	5.1
Two obelisks in the ruins at Thebes	63.3	4.5	5.1
Great obelisk at Constantinople	59.7	4.5	7.2
Obelisk in the Piazza Navona, removed from the Circus of Caracalla	54.9	2.9	4.5
Obelisk at Arles	50.1	4.5	7.4
Obelisk from the Mausoleum of Augustus, now in front of the Church of Sta. Maria Maggiore, at Rome	48.3	2.9	4.3
Obelisk in the Gardens of Sallust, according to Mercati	48.3	2.9	4.3
Obelisk at Bijije, in Egypt	42.9	2.6	4.2
Small obelisk at Constantinople, according to Gyllius	34.2	3.9	5.9
The Barberini Obelisk	30.0	2.2	3.9
Obelisk of the Villa Mattei	26.4	2.2	2.7
Obelisk in the Piazza della Rotunda	20.1	2.1	2.4
Obelisk in the Piazza di Minerva	17.6	2.0	2.6
Obelisk of the Villa Medici	16.1	1.9	2.4

DIMENSIONS OF SOME WELL-KNOWN EUROPEAN BUILDINGS.*

The body of Milan Cathedral, from the great doorway to the end of the apse, measures 148 metres and 10 centimetres, with a breadth of 57 metres. The total length of the transepts with the chapels is 87 metres. The nave is 47 metres high by 19 in width, and the total height, from the centre to the feet of the statue of the Virgin which crowns the central tower, is 108.5 metres.

The Cathedral of York, burned in 1828, and which had already been rebuilt in 1075, has a length of 142 English feet, a breadth of 105 feet at the western extremity, and 109 feet at the opposite end. The total height of the nave is 99 feet; the ceiling of the central tower is 213 feet from the ground. A window which opens at the extremity of the gallery, and which is entirely filled with stained glass, is 65 English feet in height by 32 in width.

The Cathedral of Cordova, built in the year 792 by the King Abderame, is 134 feet long and 287 wide. This church contains nine naves formed by 1,018 columns, the smallest of which are 7 feet, and the largest 11 feet and 3 inches high.

The Escurial, begun in 1557, to which was given the form of a gridiron, in honor of St. Lawrence, is 51 feet in height and 637 feet in length.

In the Alhambra at Granada, an ancient Moorish fortress, the Lion Court is 100 feet square.

The Church of St. Denis, near Paris, is 335 feet long by 90 feet high. It was built in 1152 by Suger.

The famous Column of the Grand Army on the Place Vendôme, Paris, is 136 feet high.

The Church of St. Geneviève, at Paris, to-day transformed into the Panthéon, is one of the most remarkable structures by reason of the vastness of its proportions. The diameter of the dome is 68 feet. The 32 columns which surround it are 31 feet in height, and the highest point of the edifice is 237 feet from the sidewalk.

The Cathedral at Rheims, which Stendhal considers one of the most beautiful churches in France, was built in 840, and measures 430 feet in length by 110 in height

The Cathedral at Strasburg, which is perhaps the only purely Gothic monument on the Continent of Europe, was finished in 1275. The first stone was laid in 1015. The tower, finished in 1439, is

* Taken from an article on Milan Cathedral, published in the *American Architect*, August 25, 1888.

without contradiction, the highest bit of masonry which exists in Europe. Its height is 426 feet; width of nave, 43 feet; length, 145 feet, inside measurements.

The tower of St. Etienne at Vienna is 414 feet high, four feet less than that at Strasburg.

The tower of St. Michael at Hamburg is 390 feet.

The famous tower of Pisa measures 193 feet, but it leans toward the south about 12 feet, which gives it a mean inclination of six feet in the hundred.

St. Sophia, at Constantinople, measures 270 feet in length by 240 feet in width, from north to south. The height of the dome above the level of the ground is only 165 feet.

The towers of Notre Dame, at Paris, measure 240 feet in height. The total length of this church is 409 feet. Its interior width at the crossing is 150 feet; the width of the nave is 40 feet.

The Church of St. Paul, at London, is 500 feet in length by 169 feet in width. The height of the dome is 319 feet.

St. Peter's, at Rome; total length, including the portico and thickness of the walls, is 660 feet. The foundation walls are 21 feet and 7 inches thick. The walls of the peristyle are 8 feet and 9 inches thick, and the peristyle is 39 feet and 3 inches in width. The interior length of the crossing of St. Peter's is 98 feet. The interior width of the nave, without the aisles and chapels, is 82 feet. The total height from the floor to the summit of the cross which surmounts the dome is 408 feet. The height of the dome under the key-stone is 249 feet. The interior height of the façade is 259 feet.

DIMENSIONS OF THE GRAND OPERA HOUSE, PARIS.

Superficial area, 37,317 square feet; and cubical contents, 428,660 metres.

The width of the façade is 230 feet.

Greatest width of building, 408 feet.

Height above the ground level, 184 feet.

From foundation to summit, 266 feet.

No less than fifteen eminent painters, fifty-six eminent sculptors, besides nineteen sculptors of ornament, were engaged on the external and internal decorations.

M. Garnier, the architect, gave his entire and unremitting attention to it, and, with the aid of his assistants, produced more than 80,000 drawings. The building was in course of construction for thirteen years.

DESCRIPTION OF NOTABLE AMERICAN BUILDINGS.

THE UNITED STATES CAPITOL.

[From "King's Hand-book of Washington."]

The site of the building is 89½ feet above ordinary low tide in the Potomac. Entire length of building, 751 feet: greatest depth (breadth of wings), 324 feet; area covered by building, 3½ acres. The central building is 352 feet long; corridors, 44 feet long; wings, 143 feet front, 239 feet deep, exclusive of porticos and steps. Central building is freestone from quarries about 40 miles below Washington.

This is painted white. The wings are of white marble from Lee, Mass. Appropriations made by Congress from 1800 to date for the erection and modelling of the Capitol amount to $15,000,000.

Dome designed by T. U. Walter, to replace a smaller one removed in 1856. Exterior height crest of statue above base line, 307½ feet; top of lantern above balustrade of building, 218 feet; height of Statue of Freedom on the apex, 19½ feet; diameter of dome, 135½ feet.

The dome rests on an octagonal base 93 feet above the basement floor, and as it leaves the top line of the building consists of a peristyle, 124 feet in diameter, of 36 iron-fluted columns 27 feet high, and weighing six tons each.

The lantern is 15 feet in diameter and 50 feet high.

The weight of iron in the superstructure of the dome is 8,009,200 pounds. This rests on a substructure of masonry and 40 interior massive stone columns supporting heavy groined arches, upon which also rests the pavement of the Rotunda.

Height from floor of Rotunda to canopy, 180 feet; diameter of Rotunda, 96 feet.

The canopy consists of an inner shell of iron ribs and lathing, laid with plaster suitable for frescoing. It is 65⅓ feet in diameter and 21 feet vertical height.

Supreme Court Room.—Seventy-five feet long, 45 feet wide, and 45 feet high.

Hall of Representatives.—Length, 139 feet: width, 93 feet; height, 36 feet; floor, 115 feet by 67 feet. Galleries will seat about 2,500 persons.

The ceiling of the hall is of cast iron, panelled, painted, and gilded, and highly enriched with gilt mouldings. The panels are filled with glass, with stained centre-pieces representing the arms of the States. Above the ceiling is the illumination loft, with 1,500 gas-jets, for lighting the hall for night sessions.

Senate Chamber.—Length, 113¼ feet ; width, 80¾ feet ; height, 39 feet.

Floor is 83 feet long, 51 feet wide. Galleries seat 1,200 persons. The ceiling is of iron with glass panels, lighted same as Representatives Hall.

Treasury Building.—Dimensions : Four hundred and sixty-eight feet north to south, 264 feet east to west ; inclusive of porticos and steps, 582 feet by 300 feet. Cost, $6,000,000.

Architects—Robert Mills, T. U. Walter, Young, Rogers, and A. B. Mullett.

State, War, and Navy Building.—A. B. Mullett, architect. Extreme dimensions north to south, 567 feet ; east to west, 342 feet ; exclusive of projection, 471 feet north to south, and 253 feet east to west. Cost, $5,000,000.

New City Hall, Philadelphia; John McArthur, jun., architect.

Dimensions of Building.

From north to south	486 feet 6 inches.
" east to west	470 feet.
Area	4½ acres.
Number of rooms in building	520.
Total amount of floor-room	14½ acres.
Height of main tower	537 feet 4 inches.
Width at base	90 feet.
Centre of clock-face above pavement	361 feet.
Diameter of clock-face	20 feet.

State Capitol, Hartford, Conn.; R. M. Upjohn, architect, New-York City.

Exterior is of marble; building is of fireproof construction, with brick and iron floors.

Dimensions of Building.

Length	296 feet.
Depth	199 feet.
Height to top of roof	99 feet.
Height to top of figure on dome,	256 feet.
Senate chamber	50 feet × 40 feet, 35 feet high.
Representatives' hall	84 feet × 56 feet, 48 feet high.
Supreme Court-room	50 feet × 31 feet, 35 feet high.

Cost of building, $2,500,000.00.

The Washington Monument, at Washington, D.C., is 555 feet 5 inches high, and has a base of 55 feet, with an entasis of 1 foot in every 34 in height. The monument is faced with white marble and backed with blue granite to the height of 452 feet; above that the walls are entirely of marble. The average settlement of the structure at each corner is 1.7 inches. The monument is a simple plain obelisk with no embellishments whatever.

The weight of the monument is 80,470 tons, or 3.6 tons per square foot; the area covered by the foundation being 22,400 square feet.

The corner-stone of the monument was laid July 4, 1848, and the cap-stone was set Dec. 6, 1884.

Metropolitan Opera-House, New York; J. C. Cady, New York, architect.

The building fills a square 200 × 260 feet; the size of the auditorium is 85 feet 8 inches × 95 feet 6 inches; the stage is 90 feet × 101 feet, and 150 feet from top to bottom; the seating capacity is 350 ; there are 5 stories of balconies.

The trusses used for roofing the auditorium and stage are 8 panelled Belgian trusses, having a span in great part of 106 feet. They are 13 feet from centres over the auditorium, and 8 feet from centres over the stage, where they have to carry the weight of the rigging-loft and the great fire-tank, in addition to the roofing. The feet of the trusses on one side are mounted on carriages to allow for contraction and expansion. They are secured by lines of sway braces, while purlins of angle-irons running between them receive the building-blocks, which in turn receive the slating. Under the ridge of the stage roof is suspended a fire-tank of boiler-iron resembling an ordinary boiler; its length is 78 feet. It was built in its position, and tubes were built in at intervals to allow members of the roof-trusses to pass through it. Underneath the whole is a large pan to receive any possible leakage.

This tank supplies the automatic sprinklers which guard the whole stage area, and also the various lines of fire-hose.

The truss over the proscenium opening has a span of 86 feet, is 76 feet above the stage, and carries a brick wall 40 feet in height. This wall is stayed, not only by the roof masses, but by a series of compensating braces and ties.

The stage supports are of iron, instead (as usually) of wood. They are made in sections easily taken apart to admit of any desired change in the stage or the space under it. There are over 3000 separate pieces of iron-work in this part of the structure.

The cost of the building proper was $951,323.41 ; cost of heating, ventilation, seating, decoration, carpets, and furniture, $119,819.56; cost of scenery, costumes, properties, music library, etc., $142,500.00.

The Madison Square Garden, New York City.—Messrs. McKim, Mead, and White, architects. This building covers the block bounded by East Twenty-seventh Street, Fourth Avenue, Twenty-sixth Street, and Madison Avenue.

It combines an immense amphitheatre, a restaurant (80 x 90 feet), a ball-room, a concert hall, an open-air roof garden, and a theatre.

The amphitheatre is an enormous room, 310 x 194 feet and 80 feet high, with an arena containing 30,000 square feet. The room is semicircular at each end, and is provided with permanent seats for 7,800 people, with sufficient standing space left to give room for a total of 15,000 persons. This vast arena, covered by the immense roof without central support, is entirely open and free from side to side and from end to end. For summer performances the roof can be opened by machinery.

The theatre has a seating capacity of about 1,200, with standing room for 400 more.

The open-air garden extends over the roof along the Madison Avenue front. It will hold from 3,000 to 5,000 people.

The building is surmounted by an immense tower 300 feet high.

AUDITORIUM BUILDING, CHICAGO, ILL., 1887–89.

ADLER & SULLIVAN, ARCHITECTS.

The Auditorium Building includes:

1. *The Auditorium.*—Permanent seating capacity, over 4,000 ; for conventions, etc. (for which the stage will be utilized), about 8,000. Contains the most complete and costly stage and organ in the world.

2. *Recital Hall.*—Seats over 500.

3. *Business Portion* consists of stores and 136 offices, part of which are in the tower.

4. *Tower Observatory*, to which the public are admitted. U. S. Signal Service occupies part of 17th, 18th, and 19th floors of tower.

Above four departments of the building are managed by Chicago Auditorium Association.

5. *Auditorium Hotel* has 400 guest rooms. The grand dining-room (175 feet long) and the kitchen are on the top floor. The magnificent Banquet Hall is built of steel, on trusses, spanning 120 feet over the Auditorium.

Area covered by building, about one and one-half acres.

Total street frontage (fronting Congress St., Michigan and Wabash Aves.), 710 feet.

Height of main building (10 stories), 145 feet.

Height of tower above main building (8 floors), 95 feet.

Height of lantern tower above main tower (2 floors), 30 feet.

Total height, 270 feet.

Size of tower, 70 × 41 feet; the foundations cover about two and one-half times larger area.

Weight of entire building, 110,000 tons.

Weight of tower, 15,000 tons.

Exterior material: First and second stories, granite; balance of building, Bedford stone.

Cost of building, $3,200,000.

THE LONGEST BRIDGES IN THE WORLD.

["Engineering News."]

Forth Bridge, 9,100 feet.

Montreal Bridge, over the St. Lawrence, 8,791 feet.

The Baltimore & Ohio Bridge, at Havre de Grace, 6,000 feet.

Brooklyn Bridge, over the East River, 5,989 feet.

Wooden bridge at Columbia, Pa., 5,366 feet.

Monongahela Bridge, near Homestead, 5,300 feet.

Louisville Railroad Bridge, over the Ohio, 5,218 feet.

Volga, over the Syzran, Russia, 4,947 feet.

Moerdyck, Holland, 4,927 feet.

Dnieper, near Jékaterinoslaw, Russia, 4,213 feet.

Cincinnati Southern Railroad, over the Ohio, 3,950 feet.

Kiev, over the Dnieper, 3,607 feet.

Dauphin Bridge, over the Susquehanna, 3,590 feet.

Barrage Bridge, Delta of the Nile, 3,353 feet.

Havre de Grace Bridge, over the Susquehanna, 3,271 feet.

Kronprinz Rudolph, over the Danube at Vienna, 3,236 feet.

Dnieper, near Krementchong, Russia, 3,250 feet.

Brommel, over the Meuse, Holland, 3,030 feet.

Plattsmouth Bridge, over the Missouri, 3,000 feet.

Two bridges of Rotterdam, over the Meuse, 2,833 feet.

Quincy Bridge, over the Mississippi, 2,847 feet.

St. Louis Bridge, over the Mississippi, 2,574 feet.

Omaha Bridge, over the Missouri, 2,750 feet.

Saint-Esprit, over the Rhone, France, 2,460 feet.

Kuilmbourg, over the Rhine, Holland, 2,347 feet.

Cincinnati, over the Ohio, 2,233 feet.

Keokuk, Ia., over the Mississippi, 2,008 feet.

Chaumont Viaduct, valley of the Suize, France, 2,000 feet.
Menai, England, 1,957 feet.

The Brooklyn Bridge (between New-York City and Brooklyn).

The following statistics relating to the construction of the Brooklyn Bridge are taken from "The Boston Herald :"—

Size of New-York *caissôn*, 102 feet by 172 feet.
Size of Brooklyn *caissôn*, 102 feet by 168 feet.
New-York tower contains 46,945 cubic yards of masonry.
Brooklyn tower contains 38,214 cubic yards of masonry.
Length of river-span, 1595 feet 6 inches.
Length of each land-span, 930 feet.
Length of Brooklyn approach, 971 feet.
Length of New-York approach, 1562 feet 6 inches.
Total length of bridge, 5989 feet.
Width of bridge, 86 feet.
Number of cables, 4.
Diameter of each cable, 15¾ inches.
Weight of four cables, inclusive of wrapping-wire, 3538½ tons.
Ultimate strength of each cable, 12,200 tons.
Weight of wire, nearly 11 feet per pound.

Each cable contains 5296 parallel, not twisted, galvanized steel oil-coated wires, closely wrapped to a solid cylinder 15¾ inches in diameter.

Size of towers at high-water line, 59 feet by 140 feet.
Size of towers at roof-course, 53 feet by 136 feet.
Total height of towers above high water, 278 feet.

Clear height of bridge in centre of river-span above high water at 90° F., 135 feet.

Height of floor at towers above high water, 119 feet 3 inches.
Grade of roadway, 3¼ feet in 100 feet.
Size of anchorages at base, 119 feet by 129 feet.
Size of anchorages at top, 104 feet by 117 feet.
Height of anchorages, 89 feet front, 85 feet rear.
Weight of each anchor-plate, 23 tons.

OTHER NOTABLE BRIDGES.

The following bridges are notable either from their size or historical connection :

The Lagong Bridge, built over an arm of the China Sea, is 5 miles long, with 300 arches of stone, 70 feet high and 70 feet broad, and each pillar supporting a marble lion 21 feet in length. Its cost is unknown, but much exceeds that of the Forth Bridge.

The new London Bridge is constructed of granite, from the designs of L. Rennie, and considered amongst the finest specimens of bridge architecture. It was commenced in 1824, and completed in 7 years, at a cost of about $7,500,000.

The Bridge of Sighs, at Venice, over which the condemned prisoners were transported from the Judgment Hall to the place of their execution, was built in the Armada year, 1588.

The Bridge of the Holy Trinity, at Florence, consists of three beautiful elliptical arches of white marble, and stands unrivalled as a work of art. It is 322 feet long, and was completed in 1569.

The Niagara Suspension Bridge was built in 1852–1855. It is 245 feet above high water, 821 feet long, and the strength is estimated at 12,000 tons.

The Rialto, at Venice, said to have been built from the designs of Michael Angelo, consists of a single marble arch, 98 feet 6 inches long, and was completed in 1589.

The Britannia Bridge crosses the Menai Straits, Wales, at an elevation of 103 feet above high water. It is entirely of wrought iron, 1,511 feet long, and was finished in 1850. Cost, $3,000,000.

The oldest bridge in England is a triangular bridge at Croyland, in Lincolnshire, which is said to have been erected about A.D. 868. It is formed of 3 semi-arches, whose bases stand in the circumference of a circle, equidistant from each other, and uniting at the top.

Clifton Suspension Bridge, near Bristol, has a span of 703 feet, and a height of 245 feet above the water. The carriageway is 20 feet wide and footway 5½ feet wide. Cost, $500,000.

Coalbrookdale Bridge, over the Severn, has the reputation of being the first cast-iron bridge built in England. It was erected in 1779. It consists of one arch 100 feet wide. Total weight, 378½ tons.

The Tower Bridge over the Thames—not yet completed—will be a notable bridge. Its centre arch is on what is known as the "bascule" principle, to be opened by raising two leaves, so as to allow ships to pass, and having, when opened, a footbridge above, available for foot passengers 135 feet above high water. The following will, if carried out, take rank among the notable bridges of the world, namely:

A bridge across the Danube, 20 miles in length, to be constructed by the Roumanian Government, between Dudesci and Tchernavoda.

A bridge across the Hudson River, between New York and the north New Jersey shore, with a span of 2,860 feet, and, therefore, far exceeding the very wide span of the Forth Bridge.

A bridge across the Straits of Messina 2½ miles in length, connecting Sicily and Italy.

A bridge across the Bosphorus, with a span of 2,550 feet, to unite European and Asiatic Turkey.

A bridge across the English Channel, about 24 or 25 miles in length.

LEAD MEMORANDA.

For roofs and gutters use 7-pound lead.

For ships and ridges use 6-pound lead.

For flashings use 4-pound lead.

Gutters should have a fall of at least one inch in 10 feet.

No sheet of lead should be laid in greater length than ten or twelve feet without a drip to allow of expansion.

A pig of lead is about three feet long, and weighs from a hundred-weight and a fourth to a hundred-weight and a half.

Spanish pigs are about a hundred-weight.

Joints to lead pipes require a pound of solder for every inch in diameter.

WEIGHT OF WROUGHT-IRON AND STEEL.

General Rules for determining the Weight of any Piece of Wrought-Iron.

One cubic foot of wrought-iron	480	lbs.
One square foot one inch thick	$\frac{480}{12}$ or 40	lbs.
One square inch one foot long	$\frac{40}{12}$ or $3\frac{1}{3}$	lbs.
One square inch one yard long	$3\frac{1}{3} \times 3$ or 10	lbs.

Thus it appears that the weight of any piece of wrought-iron in pounds per yard is equal to ten times its area in square inches.

Example.—The area of a bar 4 inches × 1 inch = 4 square inches, and its weight is 40 lbs. per yard.

For round iron, the weight per foot may be found by taking the diameter in quarter-inches, squaring it, and dividing by 6.

Example.—What is the weight of 2-inch round iron ?

$$2 \text{ inches} = 8 \text{ quarter-inches.} \quad 8^2 = 64.$$

$$\frac{64}{6} = 10\tfrac{2}{3} \text{ lbs. per foot of 2-inch round.}$$

Example.—What is the weight of $\frac{3}{4}$-inch round iron ?

$$\tfrac{3}{4}\text{-inch} = 3 \text{ quarter-inches.} \quad 3^2 = 9.$$

$$\frac{9}{6} = 1\tfrac{1}{2} \text{ lbs. per foot of } \tfrac{3}{4}\text{-inch round.}$$

The above rules are very convenient, and enable mental calculations of weight to be quickly obtained with accuracy.

Steel.—To find the weight of a steel bar, first determine what the weight would be if of wrought-iron, and then add 2 per cent.

WEIGHT PER FOOT OF FLAT, SQUARE, AND ROUND WROUGHT-IRON.

For steel add 2 per cent.

Thickness or Diameter.		Weight of a square foot, in lbs.	Weight per Foot.	
In inches.	In decimals of a foot.		Square bar, in lbs.	Round bar, in lbs.
1/32	0.0026	1.263	0.0033	0.0026
1/16	0.0052	2.526	0.0132	0.0104
3/32	0.0078	3.789	0.0296	0.0233
1/8	0.0104	5.052	0.0526	0.0414
5/32	0.0130	6.315	0.0823	0.0646
3/16	0.0156	7.578	0.1184	0.0930
7/32	0.0182	8.841	0.1612	0.1266
1/4	0.0208	10.100	0.2105	0.1653
9/32	0.0234	11.370	0.2665	0.2093
5/16	0.0260	12.630	0.3290	0.2583
11/32	0.0287	13.890	0.3980	0.3126
3/8	0.0313	15.160	0.4736	0.3720
13/32	0.0339	16.420	0.5558	0.4365
7/16	0.0365	17.680	0.6446	0.5063
15/32	0.0391	18.950	0.7400	0.5813
1/2	0.0417	20.210	0.8420	0.6613
9/16	0.0469	22.730	1.0660	0.8370
5/8	0.0521	25.260	1.3160	1.0330
11/16	0.0573	27.790	1.5920	1.2500
3/4	0.0625	30.310	1.8950	1.4880
13/16	0.0677	32.840	2.2230	1.7460
7/8	0.0729	35.370	2.5790	2.0250
15/16	0.0781	37.890	2.9600	2.3250
1	0.0833	40.420	3.3680	2.6450
1 1/16	0.0885	42.940	3.8030	2.9860
1 1/8	0.0938	45.470	4.2630	3.3480
1 3/16	0.0990	48.000	4.7500	3.7300
1 1/4	0.1042	50.520	5.2630	4.1330
1 5/16	0.1094	53.050	5.8020	4.5570
1 3/8	0.1146	55.570	6.3680	5.0010

WEIGHT PER FOOT OF FLAT, SQUARE, AND ROUND WROUGHT-IRON (*Continued*).

For steel add 2 per cent.

Thickness or Diameter.		Weight of a square foot, in lbs.	Weight per Foot.	
In inches.	In decimals of a foot.		Square bar, in lbs.	Round bar, in lbs.
$1\frac{7}{16}$	0.1198	58.10	6.960	5.466
$1\frac{1}{2}$	0.1250	60.63	7.578	5.952
$1\frac{5}{8}$	0.1354	65.68	8.893	6.985
$1\frac{3}{4}$	0.1458	70.73	10.310	8.101
$1\frac{7}{8}$	0.1563	75.78	11.840	9.300
2	0.1667	80.83	13.470	10.580
$2\frac{1}{8}$	0.1771	85.89	15.210	11.950
$2\frac{1}{4}$	0.1875	90.94	17.050	13.390
$2\frac{3}{8}$	0.1979	95.99	19.000	14.920
$2\frac{1}{2}$	0.2083	101.00	21.050	16.530
$2\frac{5}{8}$	0.2188	106.10	23.210	18.230
$2\frac{3}{4}$	0.2292	111.20	25.470	20.010
$2\frac{7}{8}$	0.2396	116.20	27.840	21.870
3	0.2500	121.30	30.310	23.810
$3\frac{1}{8}$	0.2604	126.30	32.890	25.830
$3\frac{1}{4}$	0.2708	131.40	35.570	27.940
$3\frac{3}{8}$	0.2813	136.40	38.370	30.130
$3\frac{1}{2}$	0.2917	141.50	41.260	32.410
$3\frac{5}{8}$	0.3021	146.50	44.260	34.760
$3\frac{3}{4}$	0.3125	151.60	47.370	37.200
$3\frac{7}{8}$	0.3229	156.60	50.570	39.720
4	0.3333	161.70	53.890	42.330
$4\frac{1}{8}$	0.3438	166.70	57.310	45.010
$4\frac{1}{4}$	0.3542	171.80	60.840	47.780
$4\frac{3}{8}$	0.3646	176.80	64.470	50.630
$4\frac{1}{2}$	0.3750	181.90	68.200	53.570
$4\frac{5}{8}$	0.3854	186.90	72.050	56.590
$4\frac{3}{4}$	0.3958	192.00	75.990	59.690
$4\frac{7}{8}$	0.4063	197.00	80.050	62.870

WEIGHT PER FOOT OF FLAT, SQUARE, AND ROUND WROUGHT-IRON (*Concluded*).

For steel add 2 per cent.

Thickness or Diameter.		Weight of a square foot, in lbs.	Weight per Foot.	
In inches.	In decimals of a foot.		Square bar, in lbs.	Round bar, in lbs.
5	0.4167	202.1	84.20	66.13
5⅛	0.4271	207.1	88.47	69.48
5¼	0.4375	212.2	92.83	72.91
5⅜	0.4479	217.2	97.31	76.43
5½	0.4583	222.3	101.90	80.02
5⅝	0.4688	227.3	106.60	83.70
5¾	0.4792	232.4	111.40	87.46
5⅞	0.4896	237.5	116.30	91.31
6	0.5000	242.5	121.30	95.23
6¼	0.5208	252.6	131.60	103.30
6½	0.5417	262.7	142.30	111.80
6¾	0.5625	272.8	153.50	120.50
7	0.5833	282.9	165.00	129.60
7¼	0.6042	293.0	177.00	139.00
7½	0.6250	303.1	189.50	148.80
7¾	0.6458	313.2	202.30	158.90
8	0.6667	323.3	215.60	169.30
8¼	0.6875	333.4	229.30	180.10
8½	0.7083	343.5	243.40	191.10
8¾	0.7292	353.6	247.90	202.50
9	0.7500	363.8	272.80	214.30
9¼	0.7708	373.9	288.20	226.30
9½	0.7917	384.0	304.00	238.70
9¾	0.8125	394.1	320.20	251.50
10	0.8333	404.2	336.80	264.50
10½	0.8750	424.4	371.30	291.60
11	0.9167	444.8	407.50	320.10
11½	0.9583	464.6	445.40	349.80
12	1 foot.	485.0	485.00	380.90

WEIGHT, PER FOOT, OF FLAT IRON.

For steel add 2 per cent.

WEIGHT OF CAST-IRON PLATES.

WEIGHT, IN POUNDS, OF CAST-IRON PLATES ONE INCH THICK.

(Calculated at 450 lbs. per cubic foot.)

Length, in inches.	Width, in Inches.									
	6	8	10	12	14	16	18	20	24	30
4	6.25	8.3	10.4	12.5	14.6	16.6	18.7	20.8	25	31
6	9.37	12.5	15.6	18.7	21.8	25.0	28.1	31.2	38	47
8	12.50	16.6	20.8	25.0	29.1	33.3	37.4	41.6	50	62
10	15.60	20.8	26.0	31.2	36.4	41.6	46.8	52.0	63	78
12	18.70	25.0	31.2	37.5	43.7	49.9	56.2	62.4	75	94
14	21.80	29.2	36.4	43.7	51.0	58.2	65.5	72.8	88	109
16	24.90	33.3	41.6	50.0	58.2	66.6	74.9	83.2	100	125
18	28.10	37.5	46.8	56.2	65.5	74.9	84.2	93.6	113	140
20	31.20	41.6	52.0	62.5	72.8	83.2	93.6	104.0	125	156
22	34.30	45.8	57.2	68.6	80.1	91.5	103.0	114.4	138	172
24	37.50	50.0	62.4	75.0	87.4	99.8	112.3	124.8	150	187
26	40.60	54.0	67.6	81.2	94.6	108.2	121.7	135.2	163	203
28	43.60	58.2	72.8	87.5	101.9	116.5	131.0	145.6	175	218
30	46.80	62.4	78.0	93.7	109.2	124.8	140.4	156.0	188	234
32	49.80	66.6	83.2	100.0	116.5	133.1	150.3	166.4	200	250
36	56.10	75.0	93.6	112.5	131.0	150.0	168.4	187.2	225	281

WEIGHT OF ROLLED LEAD, COPPER, AND BRASS, SHEET AND BAR.

Thickness or diameter, in inches.	Lead. Sheets, per square foot.	Lead. Square bars 1 foot long.	Lead. Round bars 1 foot long.	Copper. Sheets, per square foot.	Copper. Square bars 1 foot long.	Copper. Round bars 1 foot long.	Brass. Sheets, per square foot.	Brass. Square bars 1 foot long.	Brass. Round bars 1 foot long.	Thickness or diameter, in inches.
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	
1-32	1.86	0.005	0.004	1.44	0.004	0.003	1.36	0.004	0.003	1-32
1-16	3.72	0.019	0.015	2.89	0.015	0.012	2.71	0.014	0.011	1-16
3-32	5.58	0.044	0.034	4.33	0.034	0.027	4.06	0.032	0.025	3-32
1-8	7.44	0.078	0.061	5.77	0.060	0.047	5.42	0.056	0.044	1-8
5-32	9.30	0.121	0.095	7.20	0.094	0.074	6.75	0.088	0.069	5-32
3-16	11.20	0.174	0.137	8.66	0.135	0.106	8.13	0.127	0.100	3-16
7-32	13.00	0.237	0.187	10.10	0.184	0.144	9.50	0.173	0.136	7-32
1-4	14.90	0.310	0.244	11.50	0.240	0.189	10.80	0.226	0.177	1-4
5-16	18.60	0.485	0.381	14.40	0.376	0.295	13.50	0.353	0.277	5-16
3-8	22.30	0.698	0.548	17.30	0.541	0.425	16.30	0.508	0.399	3-8
7-16	26.00	0.950	0.746	20.20	0.736	0.578	19.00	0.691	0.543	7-16
1-2	29.80	1.240	0.974	23.10	0.962	0.755	21.70	0.903	0.709	1-2
9-16	33.50	1.570	1.230	26.00	1.220	0.955	24.30	1.140	0.900	9-16
5-8	37.20	1.940	1.520	28.90	1.500	1.180	27.10	1.410	1.110	5-8
11-16	40.90	2.340	1.840	31.70	1.820	1.430	29.80	1.700	1.340	11-16
3-4	44.60	2.790	2.190	34.60	2.160	1.700	32.50	2.030	1.600	3-4
13-16	48.30	3.270	2.570	37.50	2.550	1.990	35.20	2.380	1.870	13-16
7-8	52.10	3.800	2.980	40.40	2.940	2.310	37.90	2.760	2.170	7-8
15-16	56.00	4.370	3.420	43.30	3.380	2.650	40.60	3.180	2.490	15-16
1	59.50	4.960	3.900	46.20	3.850	3.020	43.30	3.610	2.840	1
1 1-8	66.90	6.270	4.920	52.00	4.870	3.820	48.70	4.570	3.600	1 1-8
1 1-4	74.40	7.750	6.090	57.70	6.010	4.720	51.20	5.640	4.430	1 1-4
1 3-8	81.80	9.370	7.370	63.50	7.280	5.720	59.60	6.820	5.370	1 3-8
1 1-2	89.30	11.200	8.770	69.30	8.650	6.800	65.00	8.120	6.380	1 1-2
1 5-8	96.70	13.100	10.30	75.10	10.200	7.980	70.40	9.530	7.490	1 5-8
1 3-4	104.00	15.200	11.90	80.80	11.800	9.250	75.90	11.100	8.680	1 3-4
1 7-8	112.00	17.500	13.70	86.60	13.500	10.600	81.30	12.700	9.970	1 7-8
2	119.00	19.800	15.60	92.30	15.400	12.100	86.70	14.400	11.300	2

WEIGHT OF ONE HUNDRED BOLTS WITH SQUARE HEADS AND NUTS.

[Hoopes & Townsend's list.]

Length under head to point.	Diameter of Bolts.								
	$\frac{1}{4}$ in.	$\frac{5}{16}$ in.	$\frac{3}{8}$ in.	$\frac{7}{16}$ in.	$\frac{1}{2}$ in.	$\frac{5}{8}$ in.	$\frac{3}{4}$ in.	$\frac{7}{8}$ in.	1 in.
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
$1\frac{1}{2}$	4.00	7.00	10.50	15.20	22.50	39.50	63.00	–	–
$1\frac{3}{4}$	4.35	7.50	11.25	16.30	23.82	41.62	66.00	–	–
2	4.75	8.00	12.00	17.40	26.15	43.75	69.00	109.00	163
$2\frac{1}{4}$	5.15	8.50	12.75	18.50	26.47	45.88	72.00	113.25	169
$2\frac{1}{2}$	5.50	9.00	13.50	19.60	27.80	48.00	75.00	117.50	174
$2\frac{3}{4}$	5.75	9.50	14.25	20.70	29.12	50.12	78.00	121.75	180
3	6.25	10.00	15.00	21.80	30.45	52.25	81.00	120.00	185
$3\frac{1}{2}$	7.00	11.00	16.50	24.00	33.10	56.50	87.00	134.25	196
4	7.75	12.00	18.00	26.20	35.75	60.75	93.10	142.50	207
$4\frac{1}{2}$	8.50	13.00	19.50	28.40	38.40	65.00	99.05	151.00	218
5	9.25	14.00	21.00	30.60	41.05	69.25	105.20	159.55	229
$5\frac{1}{2}$	10.00	15.00	22.50	32.80	43.70	73.60	111.25	168.00	240
6	10.75	16.00	24.00	35.00	46.35	77.75	117.30	176.60	251
$6\frac{1}{2}$	–	–	25.50	37.20	49.00	82.00	123.35	185.00	262
7	–	–	27.00	39.40	51.65	86.25	129.40	193.65	273
$7\frac{1}{2}$	–	–	28.50	41.60	54.30	90.50	135.00	202.00	284
8	–	–	30.00	43.80	59.60	94.75	141.60	210.70	295
9	–	–	–	46.00	64.90	103.25	153.60	227.75	317
10	–	–	–	48.20	70.20	111.75	166.70	224.80	339
11	–	–	–	50.40	76.50	120.25	177.80	261.85	360
12	–		–	52.60	80.80	128.75	189.90	278.90	382
13	–	–	–	–	86.10	137.26	202.00	295.95	401
14	–	–	–	–	91.40	145.75	214.10	313.00	[illegible]
15		–	–	–	96.70	154.25	226.20	330.05	448
16	–	–	–	–	102.00	162.75	238.30	347.10	470
17	–	–	–	–	107.30	171.00	250.40	364.15	492
18	–	–	–	–	112.60	179.50	262.60	381.20	514
19	–	–	–	–	117.90	188.00	274.70	398.25	536
20	–	–	–	–	123.20	206.50	286.80	415.30	558
Per inch add.'l	1.37	2.13	3.07	4.18	5.45	8.52	12.27	16.70	21.82

WEIGHTS OF NUTS AND BOLT-HEADS, IN POUNDS.

For calculating the weight of longer bolts.

IRON RIVETS.—WEIGHT PER HUNDRED.

Length under head.	Diameters.						
	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1
1	1.895	4.848	0.966	16.79	26.49	39.3	55.2
$1\frac{1}{8}$	2.067	5.235	10.340	17.86	27.99	41.4	57.9
$1\frac{1}{4}$	2.238	5.616	11.040	18.96	29.61	43.5	60.7
$1\frac{3}{8}$	2.410	6.003	11.730	20.03	31.13	45.6	63.4
$1\frac{1}{2}$	2.582	6.402	12.430	21.04	32.74	47.8	66.2
$1\frac{5}{8}$	2.754	6.789	13.120	22.11	34.25	49.9	68.9
$1\frac{3}{4}$	2.926	7.179	13.810	23.21	35.86	52.0	71.7
$1\frac{7}{8}$	3.098	7.566	14.500	24.28	37.37	54.1	74.4
2	3.269	7.956	15.190	25.48	38.99	56.3	77.2
$2\frac{1}{8}$	3.441	8.343	15.880	26.56	40.40	58.4	79.9
$2\frac{1}{4}$	3.613	8.733	16.570	27.65	42.11	60.5	82.7
$2\frac{3}{8}$	3.785	9.120	17.260	28.73	43.67	62.6	85.4
$2\frac{1}{2}$	3.957	9.511	17.950	29.82	45.24	64.8	88.2
$2\frac{5}{8}$	4.129	9.898	18.640	30.90	46.80	66.9	90.9
$2\frac{3}{4}$	4.301	10.290	19.330	31.99	48.36	69.0	93.7
$2\frac{7}{8}$	4.473	10.670	20.020	33.08	49.92	71.1	96.4
3	4.644	11.060	20.710	34.18	51.49	73.3	99.2
$3\frac{1}{8}$	4.816	11.440	21.400	35.27	53.05	75.4	101.9
$3\frac{1}{4}$	4.988	11.840	22.090	36.35	54.61	77.5	104.7
$3\frac{3}{8}$	5.160	12.230	22.780	37.44	56.17	79.6	107.4
$3\frac{1}{2}$	5.332	12.620	23.480	38.52	57.74	81.8	110.2
$3\frac{5}{8}$	5.504	13.010	24.170	39.60	59.30	83.9	112.9
$3\frac{3}{4}$	5.676	13.390	24.860	40.69	60.86	86.0	116.7
$3\frac{7}{8}$	5.848	13.780	25.550	41.78	62.42	88.1	119.4
4	6.019	14.170	26.240	42.87	63.99	90.3	121.2
$4\frac{1}{8}$	6.191	14.560	26.930	43.94	65.55	92.4	123.9
$4\frac{1}{4}$	6.363	14.950	27.620	45.01	67.11	94.5	126.6
100 heads.	0.519	1.74	4.14	8.10	13.99	22.27	33.15

Length of rivet required to make one head = $1\frac{1}{4}$ diameters of round bar.

NAILS AND SPIKES.

SIZE, LENGTH, AND NUMBER TO THE POUND.

Cumberland Nail and Iron Company.

TACKS.

Size.	Length.	Number to pound.	Size.	Length.	Number to pound	Size.	Length.	Number to pound.
1 oz.	$\frac{1}{8}$	16000	4 oz.	$\frac{7}{16}$	4000	14 oz.	$\frac{13}{16}$	1143
$1\frac{1}{2}$ "	$\frac{3}{16}$	10666	6 "	$\frac{1}{2}$	2666	16 "	$\frac{7}{8}$	1000
2 "	$\frac{1}{4}$	8000	8 "	$\frac{5}{8}$	2000	18 "	$\frac{15}{16}$	888
$2\frac{1}{2}$ "	$\frac{5}{16}$	6400	10 "	$\frac{11}{16}$	1600	20 "	1	800
3 "	$\frac{3}{8}$	5333	12 "	$\frac{3}{4}$	1833	22 "	$1\frac{1}{16}$	727

WEIGHT OF PLAIN CAST-IRON PIPES.

WEIGHT OF A LINEAR FOOT WITHOUT JOINTS.

Bore, in inches.	Thickness of Metal, in Inches.								
	1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
2	5.5	8.7	12.3	16.1	20.3	24.7	29.5	34.5	39.9
2½	6.8	10.6	14.7	19.2	24.0	29.0	34.4	40.0	46.0
3	7.9	12.4	17.2	22.2	27.6	32.3	39.3	45.6	52.2
3½	9.2	14.3	19.6	25.3	31.3	37.6	44.2	51.0	58.3
4	10.4	16.1	22.1	28.4	35.0	41.9	49.1	56.6	64.4
4½	11.7	18.0	24.5	31.5	38.7	46.2	54.0	62.1	70.6
5	12.9	19.8	27.0	34.5	42.3	50.5	59.9	67.7	76.7
5½	14.1	21.6	29.5	37.6	46.0	54.8	63.8	73.2	82.9
6	15.3	23.5	31.9	40.7	49.7	59.1	68.7	78.7	89.0
7	17.8	27.2	36.9	46.8	57.1	67.7	78.5	89.8	101.0
8	20.3	30.8	41.7	52.9	64.4	76.2	88.4	101.0	114.0
9	22.7	34.5	46.6	59.1	71.8	84.8	98.2	112.0	126.0
10	25.2	38.2	51.5	65.2	79.2	93.4	108.0	123.0	138.0
11	27.6	41.9	56.5	71.3	86.5	102.0	118.0	134.0	150.0
12	30.1	45.6	61.4	77.5	93.9	111.0	128.0	145.0	163.0
13	32.5	49.2	66.3	83.6	101.0	119.0	138.0	156.0	175.0
14	35.0	52.9	71.2	89.7	109.0	128.0	147.0	167.0	187.0
15	37.4	56.6	76.1	95.9	116.0	136.0	157.0	178.0	199.0
16	39.1	60.3	81.0	102.0	123.0	145.0	167.0	189.0	212.0
18	44.8	67.7	90.9	114.0	138.0	162.0	187.0	211.0	236.0
20	49.7	75.2	101.0	127.0	153.0	179.0	206.0	233.0	261.0
22	54.6	82.6	111.0	139.0	168.0	197.0	226.0	255.0	285.0
24	59.6	89.9	120.0	151.0	182.0	214.0	245.0	278.0	310.0
26	64.5	97.3	131.0	164.0	198.0	231.0	266.0	300.0	335.0
28	69.4	105.0	140.0	176.0	212.0	249.0	286.0	323.0	360.0
30	74.2	112.0	150.0	188.0	227.0	266.0	305.0	345.0	384.0

NOTE. — For each joint, add a foot to length of pipe.

WEIGHTS, PER FOOT, OF CAST-IRON PIPES IN GENERAL USE, INCLUDING SOCKET AND SPIGOT ENDS.

[Dennis Long & Co.]

Diameter.	Thickness.	Weight per foot.	Diameter.	Thickness.	Weight per foot.
2 inches.	$\frac{1}{4}$ + inch.	$6\frac{1}{4}$ lbs.	14 inches.	$\frac{7}{8}$ inch.	138 lbs.
2 "	$\frac{3}{8}$ "	$9\frac{1}{4}$ "	16 "	$\frac{1}{2}$ "	85 "
2 "	$\frac{1}{2}$ "	14 "	16 "	$\frac{5}{8}$ "	108 "
3 "	$\frac{1}{4}$ + "	11 "	16 "	$\frac{3}{4}$ "	129 "
3 "	$\frac{3}{8}$ "	$13\frac{1}{2}$ "	16 "	$\frac{7}{8}$ "	152 "
3 "	$\frac{1}{2}$ "	18 "	16 "	1 "	175 "
3 "	$\frac{5}{8}$ "	23 "	18 "	$\frac{5}{8}$ "	114 "
4 "	$\frac{3}{8}$ + "	$16\frac{1}{2}$ "	18 "	$\frac{3}{4}$ "	137 "
4 "	$\frac{1}{2}$ "	23 "	18 "	$\frac{7}{8}$ "	161 "
4 "	$\frac{5}{8}$ "	31 "	20 "	$\frac{5}{8}$ "	132 "
6 "	$\frac{3}{8}$ "	25 "	20 "	$\frac{3}{4}$ "	160 "
6 "	$\frac{1}{2}$ "	33 "	20 "	$\frac{7}{8}$ "	197 "
6 "	$\frac{5}{8}$ "	$42\frac{1}{2}$ "	20 "	1 "	215 "
6 "	$\frac{3}{4}$ "	52 "	24 "	$\frac{5}{8}$ "	159 "
8 "	$\frac{3}{8}$ "	40 "	24 "	$\frac{3}{4}$ "	190 "
8 "	$\frac{1}{2}$ "	$43\frac{1}{2}$ "	24 "	$\frac{7}{8}$ "	224 "
8 "	$\frac{5}{8}$ "	56 "	24 "	1 "	257 "
8 "	$\frac{3}{4}$ "	68 "	30 "	$\frac{3}{4}$ "	237 "
10 "	$\frac{7}{16}$ + "	50 "	30 "	$\frac{7}{8}$ "	277 "
10 "	$\frac{1}{2}$ "	54 "	30 "	1 "	319 "
10 "	$\frac{5}{8}$ "	68 "	30 "	$1\frac{1}{8}$ "	360 "
10 "	$\frac{3}{4}$ "	80 "	36 "	$\frac{7}{8}$ "	332 "
12 "	$\frac{1}{2}$ "	67 "	36 "	1 "	381 "
12 "	$\frac{5}{8}$ "	82 "	36 "	$1\frac{1}{8}$ "	429 "
12 "	$\frac{3}{4}$ "	99 "	36 "	$1\frac{1}{4}$ "	479 "
12 "	$\frac{7}{8}$ "	117 "	48 "	1 "	512 "
14 "	$\frac{1}{2}$ "	74 "	48 "	$1\frac{1}{8}$ "	584 "
14 "	$\frac{5}{8}$ "	94 "	48 "	$1\frac{1}{4}$ "	685 "
14 "	$\frac{3}{4}$ "	113 "	48 "	$1\frac{1}{2}$ "	775 "

WEIGHTS OF CAST-IRON WATER-PIPES.

In pounds, per foot run, including bells and spigots.

Diameter.	Philadelphia.[1]	Chicago.[2]	Cincinnati.[2]		Standard.[2]	Light.[2]
			Weight.	Thickness		
2 ins.	–	–	–	–	7	6
3 "	15.000	–	17	½ inch.	15	13
4 "	21.111	24.167	23	½ "	22	20
6 "	30.106	36.666	50	¾ "	33	30
8 "	40.683	50.000	65	¾ "	42	40
10 "	52.075	65.000	80	¾ "	60	55
12 "	69.162	83.333	100	¾ "	75	70
16 "	102.522	125.000	130	¾ "	–	–
20 "	147.681	–	200	⅞ "	–	–
24 "	–	250.000	224	⅞ "	–	–
30 "	–	–	300	1 "	–	–
36 "	–	450.000	430	1⅛ "	–	–

Water-pipe is usually tested to three hundred pounds' pressure per square inch before delivery, and a hammer test should be made while the pipe is under pressure.

The Philadelphia lengths for each section are, for three and four inch pipe, 9 feet; all larger sizes, 12 feet 3½ inches in length.

The Cincinnati lengths are uniform for all diameters, — 12 feet.

Chicago, same as Cincinnati.

Standard lengths are, for two-inch pipe, 8 feet, and all other sizes, 12 feet.

The thickness of the lead joint ranges from one-fourth inch on small sizes to one-half inch on the large sizes.

WEIGHTS OF LEAD AND GASKET FOR PIPE JOINTS.

[Dennis, Long, & Co.]

Diameter of pipe.	Lead.	Gasket.	Diameter of pipe.	Lead.	Gasket.
2 inches.	2.5 lbs.	0.125 lbs.	12 inches.	15 lbs.	0.250 lbs.
3 "	3.5 "	0.170 "	14 "	18 "	0.375 "
4 "	4.5 "	0.170 "	16 "	22 "	0.500 "
6 "	6.5 "	0.200 "	18 "	26 "	0.500 "
8 "	9.0 "	0.300 "	20 "	33 "	0.625 "
10 "	13.0 "	0.250 "			

[1] From Trautwine. [2] Dennis, Long, & Co., Louisville, Ky.

WEIGHT OF SQUARE CAST-IRON COLUMNS IN POUNDS PER LINEAL FOOT.

(Birkmire.)

a, b; 2a + 2b	Thickness of Metal in Inches.						
*							
12							
14							
16							
[illegible]							
20							
22							
24							
26							
28							
30							
32							
34							
36							
38							
40							
42							
44							
46							
48							
50							
52							
54							
56							
58							
60							
62	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
64	120.1	143 0	65.4	187 5	209.2	230.5	350 0
66	124 0	147.7	170.9	193.8	216.2	233 3	362 5
68	127.9	152 3	176.4	200 0	223.2	246.1	375.0
70	131.8	157.0	181.8	206.3	230 3	253 9	387 5
[illegible]	135.7	161.7	187.3	212.5	237 3	261.7	400.0
74	139.6	166.4	192 8	218 8	244.3	269 5	412 5
76	143 5	171.1	198 3	225 0	251 3	277.3	425.0
78	147 4	175.8	203 7	231 3	258.4	285.2	437.5
80	151.3	180.5	207.2	237 5	265 4	293.0	450.0

EXAMPLE.—What is the weight per lineal foot of a 12" × 16" × 1" thick column ?

Ans.—$2a + 2b = 24 + 36 = 56$. Opposite this number, under 1-inch thick metal, we find 162.5, or weight per lineal foot of a 12" × 16" × 1" thick column

NOTE.—For flanges brackets, etc., calculate the cubical contents of same and multiply by 26 ; cast-iron averaging 450 pounds per cubic foot.

* a and b = either side $2a + 2b$ = number.

WEIGHT PER LINEAL FOOT OF CIRCULAR CAST-IRON COLUMNS. (BIRKMIRE.)

Outside Diam., in Inches.	Thickness of Metal, in Inches.																
	½	⅝	¾	⅞	1	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2	2⅛	2¼	2⅜	2½
3	12.3	14.6	16.6	18.3	19.6	...	...	...	...	...	...	...	...	...	...	...	...
4	17.2	21.0	24.0	27.0	29.5	32.1	33.8	35.4	...	...	...	...	...	...	...	...	...
5	22.1	27.0	31.3	35.5	39.3	43.0	46.0	49.0	51.54	54.1	55.84	57.5	...	...	...	...	...
6	27.0	33.0	39.0	44.0	49.1	54.1	58.3	62.4	66.30	69.9	73.02	76.0	78.6	80.84	82.83	...	...
7	32.0	39.1	46.0	53.0	59.0	65.1	70.6	76.1	81.00	58.8	90.20	94.3	98.2	101.70	105.00	107.84	110.45
8	36.8	45.3	53.4	61.2	69.1	76.1	83.1	89.5	95.80	101.8	107.40	112.8	117.8	122.60	127.00	131.20	135.00
9	41.7	51.4	[illegible]	70.0	78.6	87.1	95.1	103.1	110.50	117.7	124.60	131.2	137.5	143.40	149.10	154.50	159.50
10	46.6	57.5	75.5	87.1	88.4	98.0	107.4	116.4	125.20	133.7	142.00	149.6	157.1	164.30	171.20	177.80	184.10
11	51.5	64.0	83.0	95.1	98.2	109.1	120.1	130.1	140.00	149.6	159.00	168.0	176.8	185.20	193.30	201.10	208.60
12	56.5	70.0	90.0	104.2	108.0	120.0	132.1	143.5	154.70	165.6	176.00	186.4	196.4	206.00	215.40	224.40	238.20
13	61.4	76.0	98.1	113.1	118.1	131.2	144.2	157.1	169.40	181.5	193.30	204.8	216.0	226.90	237.50	247.70	257.70
14	66.3	82.1	105.1	121.4	128.1	142.0	156.5	170.4	184.10	197.4	210.50	223.2	235.7	247.70	259.60	271.10	282.30
15	71.2	88.2	112.3	130.1	137.5	153.3	169.4	184.1	198.90	213.4	227.70	241.6	255.3	268.60	281.70	294.40	306.80
16	76.1	94.4	[illegible]	138.6	147.3	164.3	181.0	197.4	213.50	229.4	244.90	260.0	274.9	289.50	303.70	317.70	331.50
17	81.0	100.5	[illegible]	139.1	157.1	175.4	193.3	211.0	228.30	245.3	262.00	278.4	294.5	310.30	325.80	341.00	355.90
18	86.0	107.0	127.0	147.0	167.0	186.4	206.0	224.4	243.00	261.3	279.20	296.8	314.2	331.20	348.00	364.30	380.40
19	91.0	113.0	134.4	155.0	177.1	197.5	218.1	238.0	257.70	277.2	296.40	315.2	333.8	352.10	370.00	387.70	405.00
20	96.0	[illegible]	142.1	164.3	186.6	208.8	230.1	251.5	272.50	293.2	313.60	333.6	353.4	372.90	392.10	411.00	430.00
21	100.6	125.0	149.1	173.1	196.6	219.6	242.4	265.0	287.20	309.0	330.80	352.1	373.1	393.80	414.20	434.30	454.10
22	105.6	131.2	156.5	181.5	206.2	230.6	255.0	278.0	302.00	325.1	348.00	370.5	393.0	414.60	436.30	457.60	478.60
23	110.5	137.3	164.1	190.1	216.1	242.0	267.0	292.0	316.70	341.0	365.10	388.9	412.3	435.50	458.40	481.00	503.20
24	115.4	143.5	171.2	199.1	226.0	253.0	279.2	305.4	331.40	357.0	382.30	407.3	432.0	456.40	480.50	504.20	527.70

(**Note.**—The table is arranged for the weight of plain shaft. For brackets, flanges, etc., calculate the cubical contents and multiply by .26.)

WROUGHT-IRON WELDED TUBES, FOR STEAM, GAS, OR WATER.

1¼ inch and below, butt welded; proved to three hundred pounds per square inch, hydraulic pressure. | **1½ inch and above, lap welded; proved to five hundred pounds per square inch, hydraulic pressure.**

TABLE OF STANDARD DIMENSIONS (MORRIS, TASKER, & CO., Limited).

Inside diameter, in inches.	Actual outside diameter, in inches.	Thickness in inches.	Actual inside diameter, in inches.	Internal circumference, in inches.	External circumference, in inches.	Length of pipe per square foot of inside surface, in feet.	Length of pipe per square foot of outside surface, in feet.	Internal area, in inches.	External area, in inches.	Length of pipe containing one cubic foot, in feet.	Weight per foot of length, in pounds.	Number of threads per inch of screw.
1/8	0.405	0.068	0.270	0.848	1.272	14.150	9.440	0.0572	0.129	2500.00	0.243	27
1/4	0.540	0.088	0.364	1.144	1.696	10.500	7.075	0.1041	0.229	1385.00	0.422	18
3/8	0.675	0.091	0.494	1.552	2.121	7.670	5.657	0.1916	0.358	751.50	0.561	18
1/2	0.840	0.109	0.623	1.957	2.652	6.130	4.502	0.3048	0.554	472.40	0.845	14
3/4	1.050	0.113	0.824	2.589	3.299	4.635	3 637	0.5333	0.866	270.00	1.126	14
1	1.315	0.134	1.048	3.292	4.134	3.679	2.903	0.8627	1.357	166.90	1.670	11½
1¼	1.660	0.140	1.380	4.335	5.215	2.768	2.301	1.4960	2.164	96.25	2.258	11½
1½	1.900	0.145	1.611	5.061	5.969	2.371	2.010	2.0380	2.835	70.65	2.694	11½
2	2.375	0.154	2.067	6.494	7.461	1.848	1.611	3.3550	4.430	42.36	3.667	11½
2½	2.875	0.204	2.468	7.754	9.032	1.547	1.328	4.7830	6.491	30.11	5.773	8
3	3.500	0.217	3.067	9.636	10.996	1.245	1.091	7.3880	9.621	19.49	7.547	8
3½	4.000	0.226	3.548	11.146	12.566	1.077	0.955	9.8870	12.566	14.56	9.055	8
4	4.500	0.237	4.026	12.648	14.137	0.949	0.849	12.7300	15.904	11.31	10.728	8
4½	5.000	0.247	4.508	14.153	15.708	0.848	0.765	15.9390	19.635	9.03	12.492	8
5	5.563	0.259	5.045	15.849	17.475	0.757	0.629	19.9900	24.299	7.20	14.564	8
6	6.625	0.280	6.065	19.054	20.813	0.630	0.577	28.8890	34.471	4.98	18.767	8
7	7.625	0.301	7.023	22.063	23.954	0.544	0.505	38.7370	45.663	3.72	23.410	8
8	8.625	0 322	7.982	25.076	27.096	0.478	0.444	50.0390	58.426	2.88	28.348	8
9	9.688	0.344	9.001	28.277	30.433	0.425	0.394	63.6330	73.715	2.26	34.077	8
10	10.750	0.366	10.019	31.475	33.772	0.381	0.355	78.8380	90.762	1.80	40.641	8

Taper of threads, 1 to 32 on each side.

WROUGHT-IRON WELDED TUBES, EXTRA STRONG.

Table of standard dimensions.

Nominal diameter, in inches.	Actual outside diameter, in inches.	Thickness, in Inches.		Actual inside Diameter, in Inches.	
		Extra strong.	Double extra strong.	Extra strong.	Double extra strong.
1/8	0.405	0.100	–	0.205	–
1/4	0.540	0.123	–	0.294	–
3/8	0.675	0.127		0.421	–
1/2	0.840	0.149	0.298	0.542	0.244
3/4	1.050	0.157	0.314	0.736	0.422
1	1.315	0.182	0.364	0.951	0.587
1¼	1.660	0.194	0.388	1.272	0.884
1½	1.900	0.203	0.406	1.494	1.088
2	2.375	0.221	0.442	1.933	1.491
2½	2.875	0.280	0.560	2.315	1.755
3	3.500	0.304	0.608	2.892	2.284
3½	4.000	0.321	0.642	3.358	2.716
4	4.500	0.341	0.682	3.818	3.136

LAP-WELDED AMERICAN CHARCOAL IRON BOILER-TUBES.

Standard dimensions (Table of Morris, Tasker, & Co., Limited).

External diameter, in inches.	Standard thickness, in inches.	Internal diameter, in inches.	Internal circumference, in inches.	External circumference, in inches.	Length of pipe per square foot of outside surface, in ft.	Length of pipe per square foot of outside surface, in ft.	Internal area, in inches.	External area, in inches.	Weight per foot, in pounds.
1	0.072	0.856	2.689	3.142	4.460	3.819	0.575	0.785	0.708
1¼	0.072	1.106	3.474	3.927	3.455	3.056	0.960	1.227	0.900
1½	0.083	1.334	4.191	4.712	2.863	2.547	1.396	1.767	1.250
1¾	0.095	1.560	4.901	5.498	2.448	2.183	1.911	2.405	1.665
2	0.095	1.804	5.667	6.283	2.118	1.909	2.556	3.142	1.981
2¼	0.095	2.064	6.484	7.069	1.850	1.698	3.314	3.976	2.238
2½	0.109	2.283	7.172	7.854	1.673	1.528	4.094	4.909	2.755
2¾	0.109	2.533	7.957	8.639	1.508	1.390	5.039	5.940	3.045
3	0.109	2.783	8.743	9.425	1.373	1.273	6.083	7.069	3.333
3¼	0.120	3.012	9.462	10.210	1.268	1.175	7.125	8.296	3.958
3½	0.120	3.262	10.248	10.995	1.171	1.091	8.357	9.621	4.272
3¾	0.120	3.512	11.033	11.781	1.088	1.018	9.687	11.045	4.590
4	0.134	3.741	11.753	12.566	1.023	0.955	10.992	12.566	5.320
4½	0.134	4.241	13.323	14.137	0.901	0.849	14.126	15.904	6.010
5	0.148	4.720	14.818	15.708	0.809	0.764	17.497	19.635	7.226
6	0.165	5.680	17.813	18.850	0.673	0.637	25.350	28.274	9.340
7	0.165	6.657	20.914	21.991	0.574	0.545	34.805	38.484	12.435
8	0.165	7.636	23.989	25.132	0.500	0.478	45.795	50.265	14.199
9	0.180	8.615	27.055	28.274	0.444	0.424	58.291	63.617	18.002
10	0.203	9.573	30.074	31.416	0.399	0.382	71.975	78.540	22.190

In estimating the effective steam heating or boiler surface of tubes, the surface in contact with air, or gases of combustion (whether internal or external to the tubes), is to be taken.

For heating liquids by steam, superheating steam, or transferring heat from one liquid or gas to another, the mean surface of the tubes is to be taken.

AMERICAN AND BIRMINGHAM WIRE GAUGES

No. of gauge.	Thickness, in Inches. American gauge.	Thickness, in Inches. Birmingham gauge.	No. of gauge.	Thickness, in Inches. American gauge.	Thickness, in Inches. Birmingham gauge.	No. of gauge.	Thickness, in Inches. American gauge.	Thickness, in Inches. Birmingham gauge.
0000	0.4600	0.454	11	0.0907	0.120	25	0.0179	0.020
000	0.4096	0.425	12	0.0808	0.109	26	0.0160	0.018
00	0.3648	0.380	13	0.0719	0.095	27	0.0142	0.016
0	0.3248	0.340	14	0.0641	0.083	28	0.0126	0.014
1	0.2893	0.300	15	0.0570	0.072	29	0.0112	0.013
2	0.2576	0.284	16	0.0508	0.065	30	0.0100	0.012
3	0.2294	0.259	17	0.0452	0.058	31	0.0089	0.010
4	0.2043	0.238	18	0.0403	0.049	32	0.0079	0.009
5	0.1819	0.220	19	0.0359	0.042	33	0.0070	0.008
6	0.1620	0.203	20	0.0319	0.035	34	0.0063	0.007
7	0.1443	0.180	21	0.0284	0.032	35	0.0056	0.005
8	0.1285	0.165	22	0.0253	0.028	36	0.0050	0.004
9	0.1144	0.148	23	0.0225	0.025			
10	0.1019	0.134	24	0.0201	0.022			

GALVANIZED AND BLACK IRON.

Weight, in pounds, per square foot of galvanized sheet-iron, both flat and corrugated.

The numbers and thicknesses are those of the iron before it is galvanized. When a flat sheet (the ordinary size of which is from two feet to two feet and a half in width, by six to eight feet in length) is converted into a corrugated one, with corrugations five inches wide from centre to centre, and about an inch deep (the

common size), its width is thereby reduced about one-tenth part, or from thirty to twenty-seven inches; and consequently the weight per square foot of area covered is increased about one-ninth part. When the corrugated sheets are laid upon a roof, the overlapping of about two inches and a half along their sides, and of four inches along their ends, diminishes the covered area about one-seventh part more, making their weight per square foot of roof about one-sixth part greater than before. Or the weight of corrugated iron per square foot, in place on a roof, is about one-third greater than that of the flat sheets of above sizes of which it is made.

Weight of Iron per Square Foot.

No. Birmingham wire gauge.	Black.				Galvanized.			
	Flat.		Corrugated.		Flat.		Corrugated.	
	Lbs.	On roof.	Lbs.	On roof.	Lbs.	On roof.	Lbs.	On roof.
30	0.485	0.566	0.539	0.647	0.818	0.954	0.898	1.08
29	0.526	0.614	0.583	0.701	0.859	1.000	0.954	1.14
28	0.565	0.659	0.628	0.753	0.898	1.040	0.997	1.20
27	0.646	0.754	0.718	0.861	0.979	1.140	1.090	1.30
26	0.722	0.842	0.802	0.963	1.050	1.240	1.180	1.41
25	0.808	0.912	0.897	1.070	1.110	1.330	1.270	1.52
24	0.889	1.040	0.907	1.180	1.220	1.420	1.360	1.62
23	1.010	1.180	1.120	1.350	1.340	1.560	1.490	1.79
22	1.130	1.310	1.260	1.510	1.460	1.700	1.620	1.95
21	1.290	1.500	1.430	1.720	1.630	1.900	1.810	2.17
20	1.410	1.640	1.560	1.880	1.750	2.040	1.940	2.33
19	1.690	1.970	1.880	2.250	2.030	2.370	2.260	2.71
18	1.980	2.310	2.200	2.640	2.320	2.700	2.580	3.09
17	2.340	2.730	2.600	3.120	2.680	3.120	2.980	3.57
16	2.630	3.070	2.920	3.510	2.960	3.450	3.290	3.95
15	2.910	3.390	3.230	3.880	3.250	3.790	3.610	4.33
14	3.360	3.920	3.730	4.480	3.690	4.300	4.100	4.92
13	3.840	4.480	4.270	5.120	4.180	4.870	4.640	5.57

Note.—The galvanizing of sheet iron adds about one-third of a pound to its weight per square foot.

KEYSTONE BRIDGE COMPANY'S CORRUGATED IRON.

The Keystone Bridge Company's corrugations are 2.425 inches long, measured on the straight line. They require a length of iron of 2.725 inches to make one corrugation, and the depth of corrugation is [illegible] inch. One corrugation is allowed for lap in the width of the sheet, and six inches in the length, for the usual pitch of roof

of two to one. Sheets can be corrugated of any length not exceeding ten feet. The most advantageous width is thirty inches and a half, which, allowing a half-inch for irregularities, will make eleven corrugations, equal to thirty inches, or, making allowance for laps, will cover twenty-four inches and a fourth of the surface of the roof.

By actual trial it was found that corrugated iron No. 20, spanning six feet, will begin to give a permanent deflection for a load of thirty pounds per square foot, and that it will collapse with a load of sixty pounds per square foot. The distance between centres of purlins should therefore not exceed six feet, and preferably be less than this.

The following table is calculated for sheets thirty inches and a half wide before corrugating:—

Results of Test

of a corrugated sheet No. 20, two feet wide, six feet long between supports, loaded uniformly with fire-clay.

Load per square foot, in pounds.	Deflection at centre, under load, in inches.	Permanent deflection load removed, in inches.	Load per square foot, in pounds.	Deflection at centre, under load, in inches.	Permanent deflection load removed, in inches.
5	[illegible]	–	35	2[illegible]	[illegible]
10	[illegible]	–	40	2[illegible]	[illegible]
15	1	–	45	3[illegible]	1[illegible]
20	1[illegible]	–	50	4	1[illegible]
25	1[illegible]	–	55	6[illegible]	Not noted
30	1[illegible]	[illegible]	60	Broke down	" "

MEMORANDA FOR EXCAVATORS AND WELL-DIGGERS.

Excavating is generally done by the cubic yard, or square; a cubic yard being twenty-seven cubic feet; and a square is generally reckoned as eight yards, or a cube six feet by six feet by six feet.

Wells 3 feet clear diameter and ½ brick thick will require the net excavation, per foot in depth, of . . . . 11 cubic feet.

"	3 feet	6 inches	diameter,	½	brick	thick	. .	14⅙	" "
"	4 "		"	½	"	"	. .	17¾	" "
"	4 "	6 "	"	½	"	"	. .	21⅔	" "
"	5 "		"	½	"	"	. .	26	" "
"	5 "	6 "	"	½	"	"	. .	30	" "
"	6 "		"	½	"	"	. .	36	" "
"	6 "	6 "	"	1	"	"	. .	50½	" "
"	7 "		"	1	"	"	. .	56	" "
"	7 "	6 "	"	1	"	"	. .	63½	" "
"	8 "		"	1	"	"	. .	71	" "
"	8 "	6 "	"	1	"	"	. .	78½	" "
"	9 "		"	1	"	"	. .	86½	" "
"	10 "		"	1	"	"	. .	104	" "
"	10 "	6 "	"	1	"	"	. .	113	" "
"	11 "		"	1	"	"	. .	122⅔	" "
"	11 "	6 "	"	1	"	"	. .	132⅔	" "
"	12 "		"	1	"	"	. .	143⅛	" "

From 13 to 15	cubic feet of	chalk	= 1 ton weight.
17 to 19	" "	clay	
18 to 24	" "	earth	
18 to 20	" "	gravel	
19 to 25	" "	sand	

Or an average for general calculations may be taken as follows:—

14 cu. feet of	chalk	weigh 1 ton,	19 cu. feet of	gravel	weigh 1 ton.
18 " "	clay	" 1 "	22 " "	sand	" 1 "
21 " "	earth	" 1 "			

A cubic yard of earth in original position will occupy from a cubic yard and a fourth to a cubic yard and a half, when dug.

A single load of sand or loam should contain 22 cubic feet; a double load, 44 cubic feet. When buying by the load, the size of load should always be specified.

MEMORANDA FOR BRICKLAYERS.

QUANTITY OF BRICK-WORK IN BARREL-DRAINS AND WELLS,

Including wastage in clipping around the curves.

Diameter in clear.	Thickness of brick-work.	Superficial feet of brick-work in one linear yard.	Number of bricks required for one linear yard.
1 ft. 0 ins.	0 ft. 4½ ins.	16 ft. 6 ins.	115
1 " 6 "	0 " 4½ "	21 " 2 "	148
2 " 0 "	0 " 4½ "	25 " 10 "	181
2 " 0 "	0 " 9 "	33 " 0 "	462
2 " 6 "	0 " 9 "	37 " 8 "	528
2 " 6 "	1 " 1 "	43 " 2 "	906
3 " 0 "	0 " 9 "	42 " 6 "	594
3 " 0 "	1 " 1 "	47 " 10 "	1004
3 " 6 "	0 " 9 "	47 " 1 "	659
3 " 6 "	1 " 1 "	52 " 7 "	1104
4 " 0 "	0 " 9 "	51 " 10 "	725
4 " 0 "	1 " 1 "	57 " 3 "	1203
5 " 0 "	0 " 9 "	61 " 3 "	857
5 " 0 "	1 " 1 "	66 " 9 "	1402
6 " 0 "	1 " 1 "	76 " 1 "	1597
7 " 0 "	1 " 1 "	85 " 6 "	1795

NOTE.—In the Eastern States, the thickness would be four inches, eight inches, and twelve inches, instead of those given in the table, as the brick are smaller.

A load of mortar measures a cubic yard, or twenty-seven cubic feet; requires a cubic yard of sand and nine bushels of lime, and will fill thirty hods.

A bricklayer's hod, measuring 1 foot 4 inches by 9 inches by 9 inches, equals 1296 cubic inches in capacity, and contains twenty bricks.

A single load of sand and other materials equals a cubic yard, or twenty-seven cubic feet; and a double load equals twice that quantity.

A measure of lime is a single load, or cubic yard.

One thousand bricks closely stacked occupy about fifty-six cubic feet.

One thousand old bricks, cleaned and loosely stacked, occupy about seventy-two cubic feet.

One superficial foot of gauged arches requires ten bricks.

One superficial foot of facings requires seven bricks.

One yard of paving requires thirty-six *stock bricks* laid flat, or fifty-two on edge, and thirty-six *paving bricks* laid flat, or eighty-two on edge.

The bricks of different makers vary in dimensions, and those of the same maker vary also, owing to the different degrees of heat to which they are subjected in burning. The memoranda given above for brick-work are therefore only approximate. The following table gives the usual dimensions of the bricks in various parts of the country: —

Description.	Inches.	Description.	Inches.
Baltimore front .	8¼ × 4⅛ × 2⅜	Maine	7½ × 3⅜ × 2⅜
Philadelphia front,		Milwaukee . . .	8½ × 4⅛ × 2⅜
Wilmington front .		North River . .	8 × 3½ × 2¼
Trenton front . .		Trenton	8 × 4 × 2¼
Croton	8½ × 4 × 2¼	Ordinary . . .	7¾ × 3⅝ × 2¼
Colabaugh . . .	8¼ × 3⅝ × 2⅜		8¼ × 4⅛ × 2⅜

Fire-brick { Valentine's (Woodbridge, N.J.) . . . . 8⅞ × 4⅜ × 2½ ins.
Fire-brick { Downing's (Allentown, Penn.) . . . . . 9 × 4½ × 2½ ins.

The weight of the smaller sized bricks is about four pounds on the average, and of the larger about six pounds.

Dry bricks will absorb about one-fifteenth of their weight in water.

Measurement of Brickwork.

Brickwork is generally measured by the one thousand bricks, laid in the wall, and sometimes by the cubic foot. In estimating by the one thousand, the contractor figures on what the bricks will cost delivered at the site of the building, and adds to this the cost of laying in the wall, including the cost of the mortar.

The general custom in measuring the exterior brick walls of buildings is to compute the total number of brick in the wall, and then the number of face or outside brick that will be required. The difference will be the number of common brick. The outside brick generally cost more than those used for the interior, have to be culled, and the labor in laying costs more.

In measuring brickwork, it is customary to deduct all openings for doors, windows, archways, etc.; but not for small flues, ends of joists, boxes of window frames, sills, or lintels, etc., on account of the waste age of material in clipping around or filling in such parts of the work, and the increased amount of time required.

There are different methods of computing the number of brick in any given quantity of work. Some contractors will compute

the total number of cubic feet of brickwork in the building, and multiply by the number of brick contained in a cubic foot, allowing for wastage, etc. This is probably as accurate a method as can be followed. The larger number of masons, however, compute the superficial area of the walls, and multiply by the number of brick in the wall to one square foot of surface; the number, of course, depending upon the thickness of the wall.

In the *Eastern States*, the following scale will be a fair average: —

4-inch wall, or ½-brick	7½ bricks per superficial foot.			
8-inch " " 1 -brick	15	"	"	" "
12-inch " " 1½-brick	22½	"	"	" "
16-inch " " 2 -brick	30	"	"	" "
20-inch " " 2½-brick	37½	"	"	" "
24-inch " " 3 -brick	45	"	"	" "

In the Middle and Western States, the bricks are larger, and the following scale will be more correct for that section of the country: —

4½-inch wall, or ½-brick	7 bricks per superficial foot.			
9 -inch " " 1 -brick	14	"	"	" "
13 -inch " " 1½-brick	21	"	"	" "
18 -inch " " 2 -brick	28	"	"	" "
22 -inch " " 2½-brick	35	"	"	" "

And seven bricks additional for each half-brick added to thickness.

The following table shows the number of bricks in any given wall, from 4 inches to 24 inches in thickness, and for from 1 to 1000 superficial feet.

TABLE TO FIND NUMBER OF BRICKS IN A WALL

Applicable to Eastern States; for Western States, reduce by one-fifteenth.

Superficial feet of wall.	Number of Bricks to Thickness of					
	4 in.	8 in.	12 in.	16 in.	20 in.	24 in.
1	8	15	23	30	38	45
2	15	30	45	60	75	90
3	23	45	68	90	113	135
4	30	60	90	120	150	180
5	38	75	113	150	188	225
6	45	90	135	180	225	270
7	53	105	158	210	263	315
8	60	120	180	240	300	360
9	68	135	203	270	338	405
10	75	150	225	300	375	450
20	150	300	450	600	750	900
30	225	450	675	900	1125	1350
40	300	600	900	1200	1500	1800
50	375	750	1125	1500	1875	2250
60	450	900	1350	1800	2250	2700
70	525	1050	1575	2100	2625	3150
80	600	1200	1800	2400	3000	3600
90	675	1350	2025	2700	3375	4050
100	750	1500	2250	3000	3750	4500
200	1500	3000	4500	6000	7500	9000
300	2250	4500	6750	9000	11250	13500
400	3000	6000	9000	12000	15000	18000
500	3750	7500	11250	15000	18750	22500
600	4500	9000	13500	18000	22500	27000
700	5250	10500	15750	21000	26250	31500
800	6000	12000	18000	24000	30000	36000
900	6750	13500	20250	27000	33750	40500
1000	7500	15000	22500	30000	37500	45000
2000	15000	30000	45000	60000	75000	90000
3000	22500	45000	67500	90000	112500	135000
4000	30000	60000	90000	120000	150000	180000
5000	37500	75000	112500	150000	187500	225000
6000	45000	90000	135000	180000	225000	270000
7000	52500	105000	157500	210000	262500	315000
8000	60000	120000	180000	240000	300000	360000
9000	67500	135000	202500	270000	337500	405000
10000	75000	150000	225000	300000	375000	450000

APPLICATION OF TABLE. — How many bricks will there be in 9846 superficial feet of wall 16 inches thick?

Answer. — In 9000 square feet there are 270000 bricks.
" 800 " " " " 24000 "
" 40 " " " " 1200 "
" 6 " " " " 180 "

In 9846 square feet there are 295380 bricks.

TABLE OF NUMBER OF BRICKS REQUIRED IN THE SETTING OF HORIZONTAL TUBULAR BOILERS.

FURNISHED BY MR. ARTHUR WALWORTH, ENGINEER OF THE WALWORTH MANUFACTURING COMPANY, BOSTON.

The number of bricks are for double 8-inch side and rear walls, with air space between. If one of the 8-inch side walls be omitted, deduct the number of bricks in the last column.

DIAMETER OF BOILER, 24 INCHES.

Length of Boiler.	Length of Grate.					Bricks in outside wall.
	2 ft.	2 ft. 6 in.	3 ft.	3 ft. 6 in.	4 ft.	
Feet.	Bricks.	Bricks.	Bricks.	Bricks.	Bricks.	
6	2427	2407	2387	2367	2347	535
7	2728	2708	2688	2668	2648	610
8	3029	3009	2989	2969	2949	685
9	3330	3310	3290	3270	3250	760
10	3631	3611	3591	3571	3551	835
11	3932	3912	3892	3872	3852	910
Firebrick.	*127*	*143*	*159*	*175*	*191*	-

DIAMETER OF BOILER, 30 INCHES.

Length of Boiler.	Length of Grate.						Bricks in one outside wall.
	2 ft. 6 in.	3 ft.	3 ft. 6 in.	4 ft.	4 ft. 6 in.	5 ft.	
Feet.	Bricks.	Bricks.	Bricks.	Bricks.	Bricks.	Bricks.	
6	3367	3344	3321	3298	3275	3252	699
7	3755	3732	3709	3686	3663	3640	797
8	4143	4120	4097	4074	4051	4028	895
9	4531	4508	4485	4462	4439	4628	993
10	4919	4896	4873	4850	4827	4804	1091
11	5307	5284	5261	5238	5215	5192	1189
12	5695	5672	5649	5626	5603	5580	1287
13	6083	6060	6037	6014	5991	5968	1385
14	6471	6448	6425	6402	6379	6356	1483
Firebrick.	*178*	*197*	*216*	*235*	*254*	*273*	-

DIAMETER OF BOILER, 36 INCHES.

Length of Boiler.	Length of Grate.						Bricks in one outside wall.
	2 ft. 6 in.	3 ft.	3 ft. 6 in.	4 ft.	4 ft. 6 in.	5 ft.	
Feet.	Bricks.	Bricks.	Bricks.	Bricks.	Bricks.	Bricks.	
8	4296	4270	4244	4218	4192	4166	905
9	4691	4665	4639	4613	4587	4561	1005
10	5086	5060	5034	5008	4982	4956	1105
11	5481	5455	5429	5403	5337	5151	1205
12	5876	5850	5824	5798	5772	5746	1305
13	6271	6245	6219	6193	6167	6141	1405
14	6666	6640	6614	6588	6562	6536	1505
15	7061	7035	7009	6983	6957	6931	1605
16	7456	7430	7404	7378	7352	7326	1705
Firebrick.	*220*	*241*	*262*	*283*	*304*	*325*	-

Table of Bricks required in Setting Boilers (*Concluded*).

Diameter of Boiler, 42 Inches.

Length of Boiler.	Length of Grate.						Bricks in one outside wall.
	3 ft.	3 ft 6 in.	4 ft.	4 ft. 6 in.	5 ft.	5 ft. 6 in.	
Feet.	Bricks.	Bricks.	Bricks.	Bricks.	Bricks.	Bricks.	
10	5777	5749	5721	5693	5665	5637	1227
11	6216	6188	6160	6132	6104	6076	1337
12	6655	6627	6599	6571	6543	6515	1447
13	7094	7066	7038	7010	6984	6954	1557
14	7533	7505	7477	7449	7421	7393	1667
15	7972	7944	7916	7888	7860	7832	1777
16	8411	8383	8355	8327	8299	8271	1887
17	8850	8822	8994	8766	8738	8710	1997
Firebrick.	*277*	*300*	*323*	*346*	*369*	*392*	-

Diameter of Boiler, 48 Inches.

Length of Boiler.	Length of Grate.						Bricks in one outside wall.
	3 ft. 6 in.	4 ft.	4 ft. 6 in.	5 ft.	5 ft. 6 in.	6 ft.	
Feet.	Bricks.	Bricks.	Bricks.	Bricks.	Bricks.	Bricks.	
10	6721	6690	6659	6628	6597	6566	1366
11	7202	7171	7140	7109	7078	7047	1487
12	7683	7652	7621	7590	7559	7528	1608
13	8164	8133	8102	8071	8040	8009	1729
14	8645	8614	8583	8552	8521	8490	1850
15	9126	9095	9064	9033	9002	8971	1971
16	9607	9576	9545	9514	9483	9452	2092
17	10088	10057	10026	9995	9964	9932	2213
18	10569	10538	10507	10476	10445	10414	2334
Firebrick.	*313*	*337*	*361*	*385*	*409*	*433*	-

Diameter of Boiler, 54 Inches.

Length of Boiler.	Length of Grate.					Bricks in one outside wall.
	4 ft.	4 ft. 6 in.	5 ft.	5 ft. 6 in.	6 ft.	
Feet.	Bricks.	Bricks.	Bricks.	Bricks.	Bricks.	
10	7233	7200	7167	7134	7101	1455
11	7746	7713	7680	7647	7614	1585
12	8259	8226	8193	8160	8127	1715
13	8772	8739	8706	8673	8640	1845
14	9285	9252	9219	9186	9153	1975
15	9798	9765	9732	9699	9666	2105
16	10311	10278	10245	10212	10179	2235
17	10824	10791	10758	10725	10692	2365
18	11337	11304	11271	11238	11205	2495
Firebrick.	*374*	*401*	*428*	*455*	*482*	-

Measurement of Stone Work.

Stone walls are generally measured by the perch, which is 16 feet 6 inches long, 18 inches thick, and 12 inches high, and contains 24¾ cubic feet. It is generally reckoned, however, as 25 cubic feet. In some localities, 22 cubic feet, or 16 feet 6 inches long × 16 inches wide × 12 inches high, is called a perch, when measured in the wall. Occasionally stone work is measured by the cubic yard of 27 cubic feet.

Net measurement is that where all openings through the walls are deducted, and 24¾ cubic feet allowed to one perch. Gross measurement is that where no openings under one perch are deducted, and 25 cubic feet allowed to one perch. When openings are deducted, it is generally agreed to allow a compensation for plumbing and squaring the jambs, and for sills and lintels.

Stone walls less than 16 inches thick are reckoned as if 16 inches thick by masons, and over 16 inches thick each additional inch is counted. Rubble walls are sometimes measured by the cord of 128 cubic feet. Footing courses are always measured extra.

Face work of a superior kind of rubble masonry is measured separately and described.

Quoin stones of selected stones are allowed as block stone, and other dressings in a similar manner.

Walling of block stone is charged at per cubic foot, according to description, similar to ashlar prepared and set, including all beds and joints; but the face is charged extra per foot superficial, according to the way in which it may be dressed.

Granite, freestone, limestone, etc., used for trimming, is generally sold in rough blocks by the cubic foot. Ashlar, platforms, etc., are generally measured by the square foot; belt courses, strings, etc., by the lineal foot; the price depending upon the number of mouldings, etc. Marble, bluestone, and slate are sold by the square foot, the price varying according to the thickness.

DRAIN-PIPE.

There are three kinds of drain-pipe offered in the market; viz., "Salt Glazed Vitrified Clay-Pipe," "Slip Glazed Clay-Pipe," and "Cement Pipe." The name of the latter sufficiently indicates what it is without any description.

The "Slip Glazed Clay-Pipe" is made of what is known as "fire" (such as fire-brick) clay, which retains its porosity when subjected to the most intense heat. It is glazed with another kind of clay, known as "slip," which, when subjected to heat, melts, creating a

very thin glazing, which, being *a foreign substance to the body of the pipe*, is liable to wear or scale off.

"Salt Glazed Clay-Pipe" is made of a clay, which, when subjected to an intense heat, becomes vitreous or glass-like; and is glazed by the vapors of salt, the salt being thrown in the fire, thereby creating a vapor which unites chemically with the clay, and forms a glazing, which will not scale or wear off, and is impervious to the action of acids, gases, steam, or any other known substance. It unites with the clay in such a manner as to form *part of the body of the pipe*, and is therefore indestructible.

Salt-glazed pipe can only be made from clay that will vitrify, that is, when subjected to an intense heat will come to a hard, compact body, not porous. And it should be borne in mind that "slip glazing" is only resorted to when the clays are of such a nature that they will not vitrify.

The material of drain-pipes should be a hard, vitreous substance; not porous, since this would lead to the absorption of the impure contents of the drain, would have less actual strength to resist pressure, would be more affected by the frost, or by the formation of crystals in connection with certain chemical combinations, or would be more susceptible to the chemical action of the constituents of the sewerage.

"Much experience with cement sewer-pipes seems to demonstrate that they are not sufficiently uniform in quality, nor sufficiently strong and durable, to be used with confidence in any important work, whether public or private. *Sewer-pipes should be salt glazed*, as this requires them to be subjected to a much more intense heat than is needed for 'slip' glazing, and thus secures a harder material."

The standard salt glazed sewer and drain pipe manufactured by the Akron Sewer Pipe Company of Akron, O., has been found to answer all requirements, and is one of the best drain-pipes to be found in the market.

The following table gives the capacity of the different sizes of drain-pipe for different inclinations. Data for computing the amount of rain-water to be provided for over any prescribed area is also given.

CAPACITY OF PIPE.

Size of Pipe.	Gallons per Minute.							
	1½ inch fall per hundred feet.	3 inch fall per hundred feet.	6 inch fall per hundred feet.	9 inch fall per hundred feet.	12 inch fall per hundred feet.	18 inch fall per hundred feet.	2 feet fall per hundred feet.	3 feet fall per hundred feet.
3 inch.	21	30	42	52	60	74	85	104
4 "	36	52	76	92	108	132	148	184
6 "	84	120	169	206	240	294	338	414
9 "	232	330	470	570	660	810	930	1140
12 "	470	680	960	1160	1360	1670	1920	2350
15 "	830	1180	1680	2040	2370	2920	3340	4100
18 "	1300	1850	2630	3200	3740	4600	5270	6470
20 "	1760	2450	3450	4180	4860	5980	6850	8410

The maximum rainfall, as shown by statistics, is about an inch per hour (except during very heavy storms), equal to 22,633 gallons per hour for each acre, or 377 gallons per minute per acre.

Owing to various obstructions, not more than fifty to seventy-five per cent of the rainfall will reach the drain within the same hour, and allowance should be made for this fact in determining size of pipe required.

TABLE OF BOARD MEASURE.

EXPLANATION. — The length of the board is given, in feet, in the left-hand column; the width is given, in inches, in the upper row of figures; and the contents are given under the width, and opposite the length. Thus, the contents of a board 13 feet long and 7 inches wide will be found under 7, and opposite 13, and is 7 feet 7 inches.

Length, in feet.	WIDTH, IN INCHES.																
	6		7		8		9		10		11		12	13		14	
	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	feet.	ft.	in.	ft.	in.
1	0	6	0	7	0	8	0	9	0	10	0	11	1	1	1	1	2
2	1	0	1	2	1	4	1	6	1	8	1	10	2	2	2	2	4
3	1	6	1	9	2	0	2	3	2	6	2	9	3	3	3	3	6
4	2	0	2	4	2	8	3	0	3	4	3	8	4	4	4	4	8
5	2	6	2	11	3	4	3	9	4	2	4	7	5	5	5	5	10
6	3	0	3	6	4	0	4	6	5	0	5	6	6	6	6	7	0
7	3	6	4	1	4	8	5	3	5	10	6	5	7	7	7	8	2
8	4	0	4	8	5	4	6	0	6	8	7	4	8	8	8	9	4
9	4	6	5	3	6	0	6	9	7	6	8	3	9	9	9	10	6
10	5	0	5	10	6	8	7	6	8	4	9	2	10	10	10	11	8
11	5	6	6	5	7	4	8	3	9	2	10	1	11	11	11	12	10
12	6	0	7	0	8	0	9	0	10	0	11	0	12	13	0	14	0
13	6	6	7	7	8	8	9	9	10	10	11	11	13	14	1	15	2
14	7	0	8	2	9	4	10	6	11	8	12	10	14	15	2	16	4
15	7	6	8	9	10	0	11	3	12	6	13	9	15	16	3	17	6
16	8	0	9	4	10	8	12	0	13	4	14	8	16	17	4	18	8
17	8	6	9	11	11	4	12	9	14	2	15	7	17	18	5	19	10
18	9	0	10	6	12	0	13	6	15	0	16	6	18	19	6	21	0
19	9	6	11	1	12	8	14	3	15	10	17	5	19	20	7	22	2
20	10	0	11	8	13	4	15	0	16	8	18	4	20	21	8	23	4
21	10	6	12	3	14	0	15	9	17	6	19	3	21	22	9	24	6
22	11	0	12	10	14	8	16	6	18	4	20	2	22	23	10	25	8
23	11	6	13	5	15	4	17	3	19	2	21	1	23	24	11	26	10
24	12	0	14	0	16	0	18	0	20	0	22	0	24	26	0	28	0
25	12	6	14	7	16	8	18	9	20	10	22	11	25	27	1	29	2
26	13	0	15	2	17	4	19	6	21	8	23	10	26	28	2	30	4
27	13	6	15	9	18	0	20	3	22	6	24	9	27	29	3	31	6
28	14	0	16	4	18	8	21	0	23	4	25	8	28	30	4	32	8
29	14	6	16	11	19	4	21	9	24	2	26	7	29	31	5	33	10
30	15	0	17	6	20	0	22	6	25	0	27	6	30	32	6	35	0
31	15	6	18	1	20	8	23	3	25	10	28	5	31	33	7	36	2

TABLE OF BOARD MEASURE (*Continued*).

																		In.
																		11
																		10
																		9
																		8
																		7
																		6
																		5
																		4
																		3
																		2
																		1
																		0
																		11
																		10
																		9
																		8
																		7
																		6
																		5
																		4
																		3
																		2
																		1
																		0
																		11
																		10
																		9
																		8
29	36	3	38	8	41	1	43	6	45	11	48	4	50	9	53	2	55	7
30	37	6	40	0	42	6	45	0	47	6	50	0	52	6	55	0	57	6
31	38	9	41	4	43	11	46	6	49	1	51	8	54	3	56	10	59	5

Scantlings reduced to Board Measure.

EXPLANATION OF TABLE. — At the left-hand of the page will be found the length of each scantling, in feet. At the head of each of the remaining columns will be found the sizes, being the width and thickness, in inches; and opposite the given length of each will be found the contents of each scantling.

Length, in feet.	1 × 2 inches.		2 × 2 inches.		2 × 3 inches.		2 × 4 inches.		2 × 5 inches.		2 × 6 inches.	2 × 7 inches.		2 × 8 inches.	
	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	feet.	ft.	in.	ft.	in.
2	0	4	0	8	1	0	1	4	1	8	2	2	4	2	8
3		6	1	0	1	6	2	0	2	6	3	3	6	4	0
4		8	1	4	2	0	2	8	3	4	4	4	8	5	4
5		10	1	8	2	6	3	4	4	2	5	5	10	6	8
6	1	0	2	0	3	0	4	0	5	0	6	7	0	8	0
7	1	2	2	4	3	6	4	8	5	10	7	8	2	9	4
8	1	4	2	8	4	0	5	4	6	8	8	9	4	10	8
9	1	6	3	0	4	6	6	0	7	6	9	10	6	12	0
10	1	8	3	4	5	0	6	8	8	4	10	11	8	13	4
11	1	10	3	8	5	6	7	4	9	2	11	12	10	14	8
12	2	0	4	0	6	0	8	0	10	0	12	14	0	16	0
13	2	2	4	4	6	6	8	8	10	10	13	15	2	17	4
14	2	4	4	8	7	0	9	4	11	8	14	16	4	18	8
15	2	6	5	0	7	6	10	0	12	6	15	17	6	20	0
16	2	8	5	4	8	0	10	8	13	4	16	18	8	21	4
17	2	10	5	8	8	6	11	4	14	2	17	19	10	22	8
18	3	0	6	0	9	0	12	0	15	0	18	21	0	24	0
19	3	2	6	4	9	6	12	8	15	10	19	22	2	25	4
20	3	4	6	8	10	0	13	4	16	8	20	23	4	26	8
21	3	6	7	0	10	6	14	0	17	6	21	24	6	28	0
22	3	8	7	4	11	0	14	8	18	4	22	25	8	29	4
23	3	10	7	8	11	6	15	4	19	2	23	26	10	30	8
24	4	0	8	0	12	0	16	0	20	0	24	28	0	32	0
25	4	2	8	4	12	6	16	8	20	10	25	29	2	33	4
26	4	4	8	8	13	0	17	4	21	8	26	30	4	34	8
27	4	6	9	0	13	6	18	0	22	6	27	31	6	36	0
28	4	8	9	4	14	0	18	8	23	4	28	32	8	37	4
29	4	10	9	8	14	6	19	4	24	2	29	33	10	38	8
30	5	0	10	0	15	0	20	0	25	0	30	35	0	40	0
31	5	2	10	4	15	6	20	8	25	10	31	36	2	41	4
32	5	4	10	8	16	0	21	4	26	8	32	37	4	42	8

SCANTLINGS REDUCED, ETC. (*Continued*).

Length, in feet.	2 × 9 inches.		2 × 10 inches.		2 × 11 inches.		2½ × 5 inches.		2½ × 6 inches.		2½ × 7 inches.		2½ × 8 inches.		2½ × 9 inches.	
	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.	ft.	in.
2	3	0	3	4	3	8	2	1	2	6	2	11	3	4	3	9
3	4	6	5	0	5	6	3	2	3	9	4	5	5	0	5	8
4	6	0	6	8	7	4	4	2	5	0	5	10	6	8	7	6
5	7	6	8	4	9	2	5	3	6	3	7	4	8	4	9	5
6	9	0	10	0	11	0	6	3	7	6	8	9	10	0	11	3
7	10	6	11	8	12	10	7	4	8	9	10	3	11	8	13	2
8	12	0	13	4	14	8	8	4	10	0	11	8	13	4	15	0
9	13	6	15	0	16	6	9	5	11	3	13	2	15	0	16	11
10	15	0	16	8	18	4	10	5	12	6	14	7	16	8	18	9
11	16	6	18	4	20	2	11	6	13	9	16	1	18	4	20	8
12	18	0	20	0	22	0	12	6	15	0	17	6	20	0	22	6
13	19	6	21	8	23	10	13	7	16	3	19	0	21	8	24	5
14	21	0	23	4	25	8	14	7	17	6	20	5	23	4	26	3
15	22	6	25	0	27	6	15	8	18	9	21	11	25	0	28	2
16	24	0	26	8	29	4	16	8	20	0	23	4	26	8	30	0
17	25	6	28	4	31	2	17	9	21	3	24	10	28	4	31	11
18	27	0	30	0	33	0	18	9	22	6	26	3	30	0	33	9
19	28	6	31	8	34	10	19	10	23	9	27	9	31	8	35	8
20	30	0	33	4	36	8	20	10	25	0	29	2	33	4	37	6
21	31	6	35	0	38	6	21	11	26	3	30	8	35	0	39	5
22	33	0	36	8	40	4	22	11	27	6	32	1	36	8	41	3
23	34	6	38	4	42	2	24	0	28	9	33	7	38	4	43	2
24	36	0	40	0	44	0	25	0	30	0	35	0	40	0	45	0
25	37	6	41	8	45	10	26	1	31	3	36	6	41	8	46	11
26	39	0	43	4	47	8	27	1	32	6	37	11	43	4	48	9
27	40	6	45	0	49	6	28	2	33	9	39	5	45	0	50	8
28	42	0	46	8	51	4	29	2	35	0	40	10	46	8	52	6
29	43	6	48	4	53	2	30	3	36	3	42	4	48	4	54	5
30	45	0	50	0	55	0	31	3	37	6	43	9	50	0	56	3
31	46	6	51	8	56	10	32	4	38	9	45	2	51	8	58	2
32	48	0	53	4	58	8	33	4	41	0	46	7	53	4	60	1

Length, in feet.	2½ × 10 inches.		2½ × 11 inches.		2½ × 12 inches.		3 × 3 inches.		3 × 4 inches.	3 × 5 inches.		3 × 6 inches.		3 × 7 inches.	
	ft.	in.	ft.	in.	ft.	in.	ft.	in.	feet.	ft.	in.	ft.	in.	ft.	in.
2	4	2	4	7	5	0	1	6	2	2	6	3	0	3	6
3	6	3	6	11	7	6	2	3	3	3	9	4	6	5	3
4	8	4	9	2	10	0	3	0	4	5	0	6	0	7	0
5	10	5	11	6	12	6	3	9	5	6	3	7	6	8	9
6	12	6	13	9	15	0	4	6	6	7	6	9	0	10	6
7	14	7	16	1	17	6	5	3	7	8	9	10	6	12	3
8	16	8	18	4	20	0	6	0	8	10	0	12	0	14	0
9	18	9	20	8	22	6	6	9	9	11	3	13	6	15	9

SCANTLINGS REDUCED, ETC. (*Continued*).

Length in feet.	2½ × 10 inches.		2½ × 11 inches.		2½ × 12 inches.		3 × 3 inches.		3 × 4 inches.	3 × 5 inches.		3 × 6 inches.		3 × 7 inches.	
	ft.	in.	ft.	in.	ft.	in.	ft.	in.	feet.	ft.	in.	ft.	in.	ft.	in.
10	20	10	22	11	25	0	7	6	10	12	6	15	0	17	6
11	22	11	25	3	27	6	8	3	11	13	9	16	6	19	3
12	25	0	27	6	30	0	9	0	12	15	0	18	0	21	0
13	27	1	29	10	32	6	9	9	13	16	3	19	6	22	9
14	29	2	32	1	35	0	10	6	14	17	6	21	0	24	6
15	31	3	34	4	37	6	11	3	15	18	9	22	6	26	3
16	33	4	36	8	40	0	12	0	16	20	0	24	0	28	0
17	35	5	39	0	42	6	12	9	17	21	3	25	6	29	9
18	37	6	41	3	45	0	13	6	18	22	6	27	0	31	6
19	39	7	43	7	47	6	14	3	19	23	9	28	6	33	3
20	41	8	45	10	50	0	15	0	20	25	0	30	0	35	0
21	43	9	48	2	52	6	15	9	21	26	3	31	6	36	9
22	45	10	50	5	55	0	16	6	22	27	6	33	0	38	6
23	47	11	52	9	57	6	17	3	23	28	9	34	6	40	3
24	50	0	55	0	60	0	18	0	24	30	0	36	0	42	0
25	52	1	57	4	62	6	18	9	25	31	3	37	6	43	9
26	54	2	59	7	65	0	19	6	26	32	6	39	0	45	6
27	56	3	61	11	67	6	20	3	27	33	9	40	6	47	3
28	58	4	64	2	70	0	21	0	28	35	0	42	0	49	0
29	60	5	66	6	72	6	21	9	29	36	3	43	6	50	9
30	62	6	68	9	75	0	22	6	30	37	6	45	0	52	6
31	64	7	71	1	77	6	23	3	31	38	9	46	6	54	3
32	66	8	73	5	80	0	24	0	32	40	0	48	0	56	0

Length in feet.	3 × 8 inches.	3 × 9 inches.		3 × 10 inches.		3 × 11 inches.		3 × 12 inches.	4 × 4 inches.		4 × 7 inches.		4 × 6 inches.
	feet.	ft.	in.	ft.	in.	ft.	in.	feet.	ft.	in.	ft.	in.	feet.
2	4	4	6	5	0	5	6	6	2	8	3	4	4
3	6	6	9	7	6	8	3	9	4	0	5	0	6
4	8	9	0	10	0	11	0	12	5	4	6	8	8
5	10	11	3	12	6	13	9	15	6	8	8	4	10
6	12	13	6	15	0	16	6	18	8	0	10	0	12
7	14	15	9	17	6	19	3	21	9	4	11	8	14
8	16	18	0	20	0	22	0	24	10	8	13	4	16
9	18	20	3	22	6	24	9	27	12	0	15	0	18
10	20	22	6	25	0	27	6	30	13	4	16	8	20
11	22	24	9	27	6	30	3	33	14	8	18	4	22
12	24	27	0	30	0	33	0	36	16	0	20	0	24
13	26	29	3	32	6	35	9	39	17	4	21	8	26
14	28	31	6	35	0	38	6	42	18	8	23	4	28
15	30	33	9	37	6	41	3	45	20	0	25	0	30
16	32	36	0	40	0	44	0	48	21	4	26	8	32
17	34	38	3	42	6	46	9	51	22	8	28	4	34

SCANTLINGS REDUCED, ETC. (*Continued*).

Length, in feet.	3 × 8 inches. feet.	3 × 9 inches. ft.	in.	3 × 10 inches. ft.	in.	3 × 11 inches. ft.	in.	3 × 12 inches. feet.	4 × 4 inches. ft.	in.	4 × 5 inches. ft.	in.	4 × 6 inches. feet.
18	36	40	6	45	0	49	6	54	24	0	30	0	36
19	38	42	9	47	6	52	3	57	25	4	31	8	38
20	40	45	0	50	0	55	0	60	26	8	33	4	40
21	42	47	3	52	6	57	9	63	28	0	35	0	42
22	44	49	6	55	0	60	6	66	29	4	36	8	44
23	46	51	9	57	6	63	3	69	30	8	38	4	46
24	48	54	0	60	0	66	0	72	32	0	40	0	48
25	50	56	3	62	6	68	9	75	33	4	41	8	50
26	52	58	6	65	0	71	6	78	34	8	43	4	52
27	54	60	9	67	6	74	3	81	36	0	45	0	54
28	56	63	0	70	0	77	0	84	37	4	46	8	56
29	58	65	3	72	6	79	9	87	38	8	48	4	58
30	60	67	6	75	0	82	6	90	40	0	50	0	60
31	62	69	9	77	6	85	3	93	41	4	51	8	62
32	64	72	0	80	0	88	0	96	42	8	53	4	64

Length, in feet.	4 × 7 inches. ft.	in.	4 × 8 inches. ft.	in.	4 × 9 inches. feet.	4 × 10 inches. ft.	in.	4 × 11 inches. ft.	in.	4 × 12 inches. feet.	5 × 5 inches. ft.	in.	5 × 6 inches. ft.	in.
2	4	8	5	4	6	6	8	7	4	8	4	2	5	0
3	7	0	8	0	9	10	0	11	0	12	6	3	7	6
4	9	4	10	8	12	13	4	14	8	16	8	4	10	0
5	11	8	13	4	15	16	8	18	4	20	10	5	12	6
6	14	0	16	0	18	20	0	22	0	24	12	6	15	0
7	16	4	18	8	21	23	4	25	8	28	14	7	17	6
8	18	8	21	4	24	26	8	29	4	32	16	8	20	0
9	21	0	24	0	27	30	0	33	0	36	18	9	22	6
10	23	4	26	8	30	33	4	36	8	40	20	10	25	0
11	25	8	29	4	33	36	8	40	4	44	22	11	27	6
12	28	0	32	0	36	40	0	44	0	48	25	0	30	0
13	30	4	34	8	39	43	4	47	8	52	27	1	32	6
14	32	8	37	4	42	46	8	51	4	56	29	2	35	0
15	35	0	40	0	45	50	0	55	0	60	31	3	37	6
16	37	4	42	8	48	53	4	58	8	64	33	4	40	0
17	39	8	45	4	51	56	8	62	4	68	35	5	42	6
18	42	0	48	0	54	60	0	66	0	72	37	6	45	0
19	44	4	50	8	57	63	4	69	8	76	39	7	47	6
20	46	8	53	4	60	66	8	73	4	80	41	8	50	0
21	49	0	56	0	63	70	0	77	0	84	43	9	52	6
22	51	4	58	8	66	73	4	80	8	88	45	10	55	0
23	53	8	61	4	69	76	8	84	4	92	47	11	57	6
24	56	0	64	0	72	80	0	88	0	96	50	0	60	0
25	58	4	66	8	75	83	4	91	8	100	52	1	62	6
26	60	8	69	4	78	86	8	95	4	104	54	2	65	0

SCANTLINGS REDUCED, ETC. (*Continued*).

Length, in feet.	4 × 7 inches.	4 × 8 inches.	4 × 9 inches.	4 × 10 inches.	4 × 11 inches.	4 × 12 inches.	5 × 5 inches.	5 × 6 inches.
	ft. in.	ft. in.	feet.	ft. in.	ft. in.	feet.	ft. in.	ft. in.
27	63 0	72 0	81	90 0	99 0	108	56 3	67 6
28	65 4	74 8	84	93 4	102 8	112	58 4	70 0
29	67 8	77 4	87	96 8	106 4	116	60 5	72 6
30	70 0	80 0	90	100 0	110 0	120	62 6	75 0
31	72 4	82 8	93	103 4	113 8	124	64 7	77 6
32	74 8	85 4	96	106 8	116 4	128	66 8	80 0

Length, in feet.	5 × 7 inches.	5 × 8 inches.	5 × 9 inches.	5 × 10 inches.	6 × 6 inches.	6 × 7 inches.	6 × 8 inches.	7 × 7 inches.
	ft. in.	ft. in.	ft. in.	ft. in.	feet.	ft. in.	feet.	ft. in.
2	5 10	6 8	7 6	8 4	6	7 0	8	8 2
3	8 9	10 0	11 3	12 6	9	10 6	12	12 3
4	11 8	13 4	15 0	16 8	12	14 0	16	16 4
5	14 7	16 8	18 9	20 10	15	17 6	20	20 5
6	17 6	20 0	22 6	25 0	18	21 0	24	24 6
7	20 5	23 4	26 3	29 2	21	24 6	28	28 7
8	23 4	26 8	30 0	33 4	24	28 0	32	32 8
9	26 3	30 0	33 9	37 6	27	31 6	36	36 9
10	29 2	33 4	37 6	41 8	30	35 0	40	40 10
11	32 1	36 8	41 3	45 10	33	38 6	44	44 11
12	35 0	40 0	45 0	50 0	36	42 0	48	49 0
13	37 11	43 4	48 9	54 2	39	45 6	52	53 1
14	40 10	46 8	52 6	58 4	42	49 0	56	57 2
15	43 9	50 0	56 3	62 6	45	52 6	60	61 3
16	46 8	53 4	60 0	66 8	48	56 0	64	65 4
17	49 7	56 8	63 9	70 10	51	59 6	68	69 5
18	52 6	60 0	67 6	75 0	54	63 0	72	73 6
19	55 5	63 4	71 3	79 2	57	66 6	76	77 7
20	58 4	66 8	75 0	83 4	60	70 0	80	81 8
21	61 3	70 0	78 9	87 6	63	73 6	84	85 9
22	64 2	73 4	82 6	91 8	66	77 0	88	89 10
23	67 1	76 8	86 3	95 10	69	80 6	92	93 11
24	70 0	80 0	90 0	100 0	72	84 0	96	98 0
25	72 11	83 4	93 9	104 2	75	87 6	100	102 1
26	75 10	86 8	97 6	108 4	78	91 0	104	106 2
27	78 9	90 0	101 3	112 6	81	94 6	108	110 3
28	81 8	93 4	105 0	116 8	84	98 0	112	114 4
29	84 7	96 8	108 9	120 10	87	101 6	116	118 5
30	87 6	100 0	112 6	125 0	90	105 0	120	122 6
31	90 5	103 4	116 3	129 2	93	108 6	124	126 7
32	93 4	106 8	120 0	133 4	96	112 0	128	130 8

SCANTLINGS REDUCED, ETC. (*Continued*).

Length, in feet.	7 × 8 inches.	7 × 9 inches.	8 × 8 inches.	8 × 9 inches.	8 × 10 inches.	9 × 9 inches.	9 × 10 inches.	9 × 11 inches.
	ft. in.	ft. in.	ft. in.	feet.	ft. in.	feet.	ft. in.	ft. in.
2	9 4	10 6	10 8	12	13 4	13 6	10 0	16 6
3	14 0	15 9	16 0	18	20 0	20 3	22 6	24 9
4	18 8	21 0	21 4	24	26 8	27 0	30 0	33 0
5	23 4	26 3	26 8	30	33 4	33 9	37 6	41 3
6	28 0	31 6	32 0	36	40 0	40 6	45 0	49 6
7	32 8	36 9	37 4	42	46 8	47 3	52 6	57 9
8	37 4	42 0	42 8	48	53 4	54 0	60 0	66 0
9	42 0	47 3	48 0	54	60 0	60 9	67 6	74 3
10	46 8	52 6	53 4	60	66 8	67 6	75 0	82 6
11	51 4	57 9	58 8	66	73 4	74 3	82 6	90 9
12	56 0	63 0	64 0	72	80 0	81 0	90 0	99 0
13	60 8	68 3	69 4	78	86 8	87 9	97 6	107 3
14	65 4	73 6	74 8	84	93 4	94 6	105 0	115 6
15	70 0	78 9	80 0	90	100 0	101 3	112 6	123 9
16	74 8	84 0	85 4	96	106 8	108 0	120 0	132 0
17	79 4	89 3	90 8	102	113 4	114 9	127 6	140 3
18	84 0	94 6	96 0	108	120 0	121 6	135 0	148 6
19	88 8	99 9	101 4	114	126 8	128 3	142 6	156 9
20	93 4	105 0	106 8	120	133 4	135 0	150 0	165 0
21	98 0	110 3	112 0	126	140 0	141 9	157 6	173 3
22	102 8	115 6	117 4	132	146 8	148 6	165 0	181 6
23	107 4	120 9	122 8	138	153 4	155 3	172 6	189 9
24	112 0	126 0	128 0	144	160 0	162 0	180 0	198 0
25	116 8	131 3	133 4	150	166 8	168 9	187 6	206 3
26	121 4	136 6	138 8	156	173 4	175 6	195 0	214 6
27	126 0	141 9	144 0	162	180 0	182 3	202 6	222 9
28	130 8	147 0	149 4	168	186 8	189 0	210 0	231 0
29	135 4	152 3	154 8	174	193 4	195 9	217 6	239 3
30	140 0	157 6	160 0	180	200 0	202 6	225 0	247 6
31	144 8	162 9	165 4	186	206 8	209 3	232 6	255 9
32	149 4	168 0	170 8	192	213 4	216 0	240 0	264 0

Plank Measure.

Board measure is the basis of plank measure; that is, a plank *two* inches thick, and thirteen feet long, and ten inches wide, contains evidently twice as many square feet as if only *one* inch thick: therefore, in estimating the contents of any plank, we first find the contents of the surface taken one inch thick, and then, if the plank be one inch and a quarter thick, we add one-quarter of the contents to *itself*, which gives the contents (in board measure) of the plank.

Contents of Planks in Board Measure. Thickness, $1\frac{1}{4}$ Inches.

Feet long	Width, in Inches.														
	6	7	8	9	10	11	12	13	14	15	16		18	19	20
	ft.	feet	feet.	feet.	feet	feet.	feet	feet	feet.	feet.	feet.		feet.	feet.	feet.
10	6	7	8	9	10	11	12	14	15	16	17	[illegible]	19	20	21
11	7	8	9	10	11	13	14	15	16	17	18		21	22	23
12	8	9	10	11	12	14	15	16	17	19	20		22	24	25
13	8	9	11	12	14	15	16	18	19	20	22	[illegible]	24	26	27
14	9	10	12	13	15	16	17	19	20	22	23		26	28	29
15	9	11	12	14	16	17	19	20	22	23	25		28	30	31
16	10	12	13	15	17	18	20	22	23	25	27	28	30	32	33
17	11	12	14	16	18	19	21	23	25	27	28	30	32	34	35
18	11	13	15	17	19	21	22	24	26	28	30	32	34	36	37
19	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
20	13	15	17	19	21	23	25	27	29	31	33	35	37	40	42
21	13	15	17	20	22	24	26	28	31	33	35	37	39	42	44
22	14	16	18	21	23	25	27	30	32	34	37	39	41	44	46
23	14	17	19	22	24	26	29	31	34	36	38	41	43	46	48
24	15	17	20	22	25	27	30	32	35	37	40	42	45	47	50
25	16	18	21	23	26	29	31	34	36	39	42	44	47	49	52
26	16	19	22	24	27	30	32	35	38	41	43	46	49	51	54
27	17	20	22	25	28	31	34	37	39	42	45	48	51	53	56
28	17	20	23	26	29	32	35	38	41	44	47	50	52	55	58
29	18	21	24	27	30	33	36	39	42	45	48	51	54	57	60
30	19	22	25	28	3	34	37	41	44	47	50	53	56	59	62
31	19	23	26	29	32	35	39	42	45	48	52	55	58	61	65
32	20	23	27	30	33	37	40	43	47	50	53	57	60	63	67
33	21	24	27	31	34	38	41	45	48	52	55	58	62	65	69
34	21	25	28	32	35	39	42	46	50	53	57	60	64	67	71
35	22	26	29	33	36	40	44	47	51	55	58	62	66	69	73

PLANK MEASURE (*Continued*).

CONTENTS OF PLANKS IN BOARD MEASURE. THICKNESS, 1½ INCHES.

CONTENTS OF PLANKS IN BOARD MEASURE. THICKNESS, 2 INCHES.

PLANK MEASURE (*Continued*).

CONTENTS OF PLANKS IN BOARD MEASURE. THICKNESS, 2 INCHES.

Feet long	6	7	8	9	10	11	12		14	15	16	17	18	19	20
	ft.	feet	feet	feet.	feet	feet.	feet.				feet.	feet	feet.	feet.	feet.
14	14	16	19	21	23						37	40	42	44	47
15	15	18	20	23	25						40	43	45	48	50
16	16	19	21	24	27						43	45	48	51	53
17	17	20	23	26	28						45	48	51	54	57
18	18	21	24	27	30						48	51	54	57	60
19	19	22	25	29	32						51	54	57	60	63
20	20	23	27	30	33			43	47		53	57	60	63	67
21	21	25	28	32	35			46	49		56	60	63	67	70
22	22	26	29	33	37			48	51			62	66	70	73
23	23	27	31	35	38			50	54				69	73	77
24	24	28	32	36	40			52	56				72	76	80
25	25	29	33	38	42			54	58				75	79	83
26	26	30	35	39	43			56	61				78	82	87
27	27	32	36	41	45			59	63	[illegible]			81	86	90
28	28	33	37	42	47			61	65			[illegible]	84	89	93
29	29	34	39	44	48			63	68		[illegible]		87	92	97
30	30	35	40	45	50			65	70		80	85	90	95	100
31	31	36	41	47	52			67	72		83	88	93	98	103
32	32	37	43	48	53			69	75		85	91	96	101	107
33	33	39	44	50	55			72	77	83	88	94	99	105	110
34	34	40	45	51	57			74	79	85	91	96	102	108	113
35	35	41	47	53	58			76	82	88	93	99	105	111	117

CONTENTS OF PLANKS IN BOARD MEASURE. THICKNESS, 2¼ INCHES.

Feet long	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
	ft.	feet	feet	feet	feet	feet	feet.	feet	feet	feet.	feet.	feet.	feet.	feet	feet
10	11	13	15	17	19	21	23	24	26	28	30	32	34	36	38
11	12	14	17	19	21	23	25	27	29	31	33	35	37	39	41
12	13	16	18	20	23	25	27	29	32	34	36	38	41	43	45
13	15	17	20	22	24	27	29	32	34	37	39	41	44	46	49
14	16	18	21	24	26	29	32	34	37	39	42	45	47	50	53
15	17	20	23	25	28	31	34	37	39	42	45	48	51	53	56
16	18	21	24	27	30	33	36	39	42	45	48	51	54	57	60
17	19	22	26	28	32	35	38	41	45	48	51	54	57	61	64

PLANK MEASURE (*Continued*).

Contents of Planks in Board Measure. Thickness, 2¼ Inches.

...ng.	Width, in Inches.
[illegible]	[illegible]

Contents of Planks in Board Measure. Thickness, 2½ Inches.

[illegible]	[illegible]

PLANK MEASURE (*Continued*).

CONTENTS OF PLANKS IN BOARD MEASURE. THICKNESS, 2½ INCHES.

Feet long.	Width, in Inches.														
	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
	ft.	feet.	feet.	feet.	feet.	feet.	feet.	feet.	feet.	feet.	feet.	feet.	feet.	feet.	feet.
22	27	32	37	41	46	50	55	60	64	69	73	78	83	87	92
23	29	34	38	43	48	53	58	62	67	72	77	81	86	91	96
24	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100
25	31	36	42	47	52	57	63	68	73	78	83	89	94	99	104
26	32	38	43	49	54	60	65	70	76	81	87	92	98	103	108
27	34	39	45	51	56	62	68	73	79	84	90	96	101	107	113
28	35	41	47	53	58	64	70	76	82	88	93	99	105	111	117
29	36	42	48	54	60	66	73	79	85	91	97	103	109	115	121
30	37	44	50	56	63	69	75	81	88	94	100	106	113	119	125
31	39	45	52	58	65	71	78	84	90	97	103	110	116	123	129
32	40	47	53	60	67	73	80	87	93	100	107	113	120	127	133
33	41	48	55	62	69	76	83	89	96	103	110	117	124	131	138
34	42	50	57	64	71	78	85	92	99	106	113	120	128	135	142
35	44	51	58	66	73	80	88	95	102	109	117	124	131	139	146

CONTENTS OF PLANKS IN BOARD MEASURE. THICKNESS, 3 INCHES.

Feet long.	Width, in Inches.														
	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
	ft.	feet.	feet.	feet.	feet.	feet.	feet.	feet.	feet	feet.	feet.	feet.	feet.	feet.	feet.
10	15	17	20	22	25	27	30	32	35	37	40	42	45	47	50
11	16	19	22	25	27	30	33	36	38	41	44	47	49	52	55
12	18	21	24	27	30	33	36	39	42	45	48	51	54	57	60
13	20	23	26	29	33	36	39	42	46	49	52	55	59	62	65
14	21	25	28	32	35	39	42	46	49	53	56	60	63	67	70
15	22	26	30	34	38	41	45	49	52	56	60	64	68	71	75
16	24	28	32	36	40	44	48	52	56	60	64	68	72	76	80
17	25	30	34	38	43	47	51	55	60	64	68	72	77	81	85
18	27	32	36	41	45	50	54	59	63	68	72	77	81	86	90
19	29	33	38	43	48	52	57	62	67	71	76	81	86	90	95
20	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100
21	31	37	42	47	53	58	63	68	74	79	84	89	95	100	105
22	33	39	44	50	55	61	66	72	77	83	88	94	99	105	110
23	34	40	46	52	58	63	69	75	81	86	92	98	104	109	115
24	36	42	48	54	60	66	72	78	84	90	96	102	108	114	120
25	37	44	50	56	63	69	75	81	88	94	100	106	113	119	125

PLANK MEASURE (*Continued*).

CONTENTS OF PLANKS IN BOARD MEASURE. THICKNESS, 3 INCHES.

CONTENTS OF PLANKS IN BOARD MEASURE. THICKNESS, 3½ INCHES.

PLANK MEASURE (*Concluded*).

CONTENTS OF PLANKS IN BOARD MEASURE. THICKNESS, 3¼ INCHES.

NAILING MEMORANDA.

[From "Builder's Guide, and Estimator's Price-Book."]

Quantity of Nails for Different Kinds of Work.

For 1000 shingles allow	3½ to 5	pounds	4*d*.	nails, or
	3 to 3½	"	3*d*.	"
1000 laths	about 6	"	3*d*.	fine
1000 feet clapboards	" 18	"	6*d*.	box
1000 " covering boards . . .	" 20	"	8*d*.	common
1000 " " "	" 25	"	10*d*.	"
1000 " upper floors, sq. edged,	" 38	"	10*d*.	floor
1000 " " "	" 41	"	12*d*.	"
1000 " " { matched and blind nailed }	" 35	"	10*d*.	"
1000 " " " "	" 42	"	12*d*.	"
10 " partitions { studs or studding }	" 1	"	10*d*.	common
1000 " furring, 1 by 3 . . .	" 45	"	10*d*.	"
1000 " " 1 by 2 . . .	" 65	"	10*d*.	"
1000 " pine finish	" 30	"	8*d*.	finish

RELATIVE HOLDING POWER OF WIRE AND CUT NAILS.

Tests made by a committee appointed by the Wheeling Nail Manufacturers.

	Number of Nails in Pound.		Pounds required to pull Nails Out.	
	Cut.	Wire.	Cut.	Wire.
20d.	23	35	1,593	703
10d.	60	86	908	315
8d.	90	126	597	227
6d.	160	206	383	200
4d.	280	316	286	123

This test showed the relative value of a pound of each kind to be as follows :

1 lb.	of 20d.	cut nails	equals	1.40	lbs.	of wire nails.
1	" 10d.	"	"	2.01	"	"
1	" 8d.	"	"	1.87	"	"
1	" 6d.	"	"	1.49	"	"
1	" 4d.	"	"	2.06	"	"

In obtaining the above results, two tests were made of the 8d. cut nails, and four of the 8d. wire nails ; three tests each were made of the 6d. and 4d. cut nails, and 6d. and 4d. wire nails, and the average is shown.

The committee report as the result of their experiments that $1.00 of cut nails will give the same service as $1.78 in wire nails, if at the same price per pound.

Very thorough tests of the comparative holding power of wire nails and cut nails *of equal lengths and weights* were made at the United States arsenal, Watertown, Mass., in November and December, 1892, and January, 1893. Fifty-eight series of tests were made, each series comprising ten pairs cut nails and wire nails, making a total of 1,160 nails tested. From forty series, comprising forty sizes of nails driven in spruce wood, it was found that the cut nails showed an average superiority of 60.50 per cent.; the common nails showing an average superiority of 47.51 per cent., and the finishing nails an average of 72.22 per cent.

In eighteen series, comprising six sizes of *box* nails driven into

pine wood, in three ways the cut nails showed an average superiority of 99.93 per cent.

In no series of tests did the wire nails hold as much as the cut nails.

MEMORANDA FOR PLASTERERS.

Measuring Plasterers' Work.

The following paragraphs, taken from one of our leading journals, describe the usual method of measuring plasterers' work: —

"Plastering is always measured by the square yard for all plain work, by the superficial foot for all cornices of plain members, and by the linear foot for enriched or carved mouldings in cornices.

"By 'plain work' is meant straight surfaces (like ordinary walls and ceilings), without regard to the style or quality of finish put upon the job. Any panelled work, whether on walls or ceilings, run with a mould, would be rated by the foot superficial.

"Different methods of valuing plastering find favor in different portions of the country. The following general rules are believed to be equitable and just to all parties: —

"*First*, Measure on all walls and ceilings the surface actually plastered, without deducting any grounds or any openings of less extent than seven superficial yards.

"*Second*, Returns of chimney-breasts, pilasters, and all strips of plastering less than twelve inches in width, measure as twelve inches wide; and where the plastering is finished down to the base, surbase, or wainscoting, add six inches to height of walls.

"*Third*. In closets, add one-half to the measurement. Raking ceilings, and soffits of stairs, add one-half to the measurement; circular or elliptical work, charge two prices; domes or groined ceilings, three prices.

"*Fourth*, For each twelve feet of interior work done farther from the ground than the first twelve feet, add five per cent; for outside work, add one per cent for each foot that the work is done above the first twelve feet.

"Stucco-work is generally governed by the following rules; viz., mouldings less than one foot girt are rated as one foot, over one foot, to be taken superficial. When work requires two moulds to run same cornice, add one-fifth. For each internal angle or mitre, add one foot to length of cornice, and, for each external angle, add two feet. All small sections of cornice less than twelve inches long measure as twelve inches. For raking cornices, add one-half; circular or elliptical work, double price; domes and groins, three prices. For enrichments of all kinds a special price must be

charged. The higher the work is above ground, the higher the charge must be; add to the rate of five per cent for every twelve feet above the first twelve feet."

Useful Memoranda.

The average yield of *lime paste* from the best Eastern limes has been found to be 2.62 times the bulk of the unslaked lime. A barrel of good quality, well-burnt lime should make eight cubic feet of lime paste.

Careful experiments, conducted by United States engineers, have demonstrated that the average sum of voids in sharp, clean, silicious bank or pit sand, thoroughly screened, is .349 of its bulk, and that the best mortar is obtained by mixing with the sand such an amount of lime paste as will be from forty-five to fifty per cent. greater than the amount needed to fill the voids of the sand, or, in other words, by mixing *one part lime paste to two of sand.*

To each barrel, or each 200 pounds of unslaked lime, one and a half bushels of good quality, long cattle hair, well whipped and washed, should be used in the first coat on lath work, and a half bushel of hair to each barrel of lime in the brown coat, whether applied over a scratch coat or on brick, iron, or terra cotta.

The lime should be slaked not less than two weeks before the plaster is applied to the walls, and the hair should be mixed in just before using. If the hair is mixed into the mortar while the lime is *hot*, the lime will burn and rot the hair.

Sand for mortar should be angular, not too coarse or too fine, and should be free from all foreign substances, and particularly fine loam or clay. Clean river or pit sand, carefully screened, is generally considered the best for mortar.

Hair, such as is used by plasterers, is obtained from the hides of cattle, and is put up in paper bags, each bag being supposed to contain one bushel of hair when beat up. The quantity of hair to be used is sometimes designated by weight, but as it is sold by the bushel or bale, that appears to be the better measure.

Plastering on lath work is generally done in three coats. The first coat is called the *scratch* coat, and is generally made very "rich." The second coat is called the *brown* coat, and usually contains a much larger proportion of sand and only a small quantity of hair. On brick and stone walls the scratch coat is generally omitted, and the brown coat is applied directly to the brick or stone work, and of the proper thickness to receive the finish coat.

The third, or finishing, coat is designated by various terms, such as *skim coat*, *white coat*, *putty coat*, *sand-finish*, etc. The skim coat as used in the Eastern States is generally composed of lime putty and washed beach sand in equal proportions. Sand-finish, which has a rough surface resembling coarse sandpaper, is mixed in the same way, only the coarser sand and more of it is used, and it is finished with a wooden or cork-faced float.

White coating, or hard finish, generally means a composition of lime putty, plaster of Paris, and marble dust. Plaster of Paris and marble dust when used should not be mixed with the lime putty until a few moments before using, and no more should be prepared at one time than can be used up at once, as it soon "sets," after which it should not be used. The skim coat, or hard finish, should be finished with a steel trowel and wet brush. The more the work is trowelled the harder it becomes.

To obtain the *best quality* of lime plaster, the specifications should read as follows:

"The mortar for plastering to be composed of best quality wood-burned stone lime, white, slaked at least fourteen days before using, and run through a fine sieve, and to be thoroughly mixed with clean, sharp sand, free from clay, loam, or other foreign substances, in the proportion of one-third lime paste to two-thirds sand, measure for measure, to be well tempered, and have the best quality of clean, long cattle hair, well wetted, thoroughly mixed with it immediately before using, as follows:

"First coat for lath work, 1½ bushels of hair to one barrel of unslaked lime; first coat for brick and terra-cotta work, and second coat for lath work, one-half bushel of hair to one barrel of unslaked lime. First coat to be put on strong, brought to a fair surface and scratched; the second coat to be put on light and well floated with long rules to a uniform surface, straight and true; each coat to be thoroughly dry before the next is put on."

In the West 200 pounds of unslaked lime is considered the equivalent of a barrel. Rockland (Me.) lime will average 220 pounds to the barrel.

Very little plaster is mixed by measure, however, the usual custom being to mix in as much sand with the slaked lime as the mortar-mixer thinks is best, or that the plaster will stand and work well. Plaster mixed in the proportions specified above will require about 2½ casks or 500 pounds of lime, 45 cubic feet or 15 casks of sand, and 4 bushels of hair, to cover 100 yards of lath work with mortar ¾ of an inch thick

For the white coat, allow 90 pounds of lime, 50 pounds of plaster of Paris, and 50 pounds of marble dust to 100 square yards.

To lath the same area will require from 1,400 to 1,500 laths, and 10 pounds of 3*d*. nails.

Sand is usually sold by the load, which varies in different localities from 18 to 27 cubic feet.

The volume of the mortar when mixed is generally about equal to that of the sand before screening.

Improved Wall Plasters.

Owing to the difficulty of obtaining an economical and satisfactory quality of walls and ceilings by the use of the ordinary lime mortar, other and more reliable plastering materials have been invented, and are now being extensively employed, especially on the largest and most costly structures, and are giving general satisfaction.

Among the best known of these improved plasters are the Acme and Climax cement plasters, Adamant, Windsor cement dry plaster, and Rock wall plaster. The Acme and Climax cements are natural products found in certain parts of Kansas and Texas, and simply calcined. The others are composed principally of plaster of Paris with certain chemicals added. All appear to produce about the same results. The Windsor dry plaster, Adamant, and Rock plaster are mixed with the proper proportion of sand by the manufacturers, and only require being "wet up" before using. All of these materials are sold by weight. They should be used strictly in accordance with the directions furnished by the manufacturers.

Among the advantages gained by the use of these plasters are: Uniformity in strength and quality; extra hardness and toughness; freedom from pitting; saving in time required in making and drying the plaster; minimum danger from frost; less weight and moisture in the building; and greater resistance to the action of fire and water.

[illegible] 27 [illegible]

Lead 700 lbs pr. square. min. slope 4°
Zinc 150 " " 4°
Corrug. iron 300 lb " 4°
3/4" boarding 250 lb 25°

MEMORANDA FOR ROOFERS.

Slate Roofs. Least slope 25½°-30°

The pitch of a slated roof should be about one in height to four in length. The usual lap is about three inches, but it is sometimes four inches. Each slate should be fastened by two *4d* or *3d* slate-nails, either of galvanized iron, copper, or zinc. On roofs of gas-houses the nails should be of copper or yellow-metal.

A square of slate is one hundred superficial feet, allowances being made for the trouble of cutting the slates at the hips, eaves, round chimneys, etc. The sides and bottom edges of the slates should be trimmed, and the nail-holes punched as near the head as possible. They should be sorted in sizes, when they are not all of one size, and the smallest placed near the ridge. The thickness of slates varies from three-sixteenths to five-sixteenths of an inch, and their weight from 2.6 to 4.53 pounds per square foot.

Elastic Cement.—In first-class work, the top course of slate on ridge, and the slate for two to four feet from all gutters, and one foot each way from all valleys and hips, should be bedded in elastic cement.

Roofing-Paper.—Roof-boards should be covered with one or two thicknesses of tarred felt roofing-paper, before the slate are laid. No dry or rosin-sized felt should be used on roofs.

Flashings.—By "flashings" are meant pieces of tin, zinc, or copper, laid over slate, and up against walls, chimneys, copings, etc.

Counter-flashings are of lead or zinc, and are laid between the courses in brick, and turned down over the flashings. In flashing against stone-work, grooves or reglets often have to be cut to receive the counter-flashings.

Close and Open Valleys.—*A close valley* is where the slate are mitred and flashed in each course, and laid in cement. In such valleys no metal can be seen. Close valleys should only be used for pitches above forty-five degrees.

An open valley is where the valley is formed of sheets of copper or zinc fifteen or sixteen inches wide, and the slate laid over these.

Rule for computing the Number of Slates in a Square.

Subtract three inches, or the amount of head-cover, from the length of the slate, multiply the remainder by the width, and divide by two. This will give the number of square inches covered per slate; divide 14,400 (the number of square inches in a square) by the number so found, and the result will be the number of slates required.

The following table gives the number of slates per square for the usual sizes, allowing three inches for head-cover: —

NUMBER OF SLATES PER SQUARE.

Size, in inches.	Pieces per square.	Size, in inches.	Pieces per square.	Size, in inches.	Pieces per square.
6 × 12	533	8 × 16	277	12 × 20	141
7 × 12	457	9 × 16	246	14 × 20	121
8 × 12	400	10 × 16	221	11 × 22	137
9 × 12	355	9 × 18	213	12 × 22	126
7 × 14	374	10 × 18	192	14 × 22	108
8 × 14	327	12 × 18	160	12 × 24	114
9 × 14	291	10 × 20	169	14 × 24	98
10 × 14	261	11 × 20	154	16 × 24	86

The weight of slate per cubic foot is about 174 pounds, or, per square foot of various thicknesses, as follows: —

Thickness, in inches	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$
Weight, in pounds	1.81	2.71	3.62	5.43	7.25

The weight of slating laid per square foot of surface covered will, of course, depend on the size used. The weight of 10 by 18 slate, three-sixteenths of an inch thick, for example, per square foot of roof, would be 5.86 pounds.

An experienced roofer will lay, on an average, two squares of slate in ten hours.

Ordinary roofing-paper weighs about fifteen pounds per square, and averages about fifty pounds in a roll.

At the present time [1884] the additional cost of laying slate in elastic cement varies from thirteen to fifteen per cent.

Comparative Cost of Different Sizes of Roofing-Slate.

The following table shows the prices for No. 1 *Monson* (*Maine*) roofing-slates delivered on wharf in Boston, May 20, 1885. It will be seen that the *medium sizes*, such as 16 × 10, 16 × 8, 18 × 10, cost the most; and, as the sizes increase or diminish from these, the price decreases. The price of *Brownville* (*Maine*) slates are in all cases $1 per square *more* than the Monson slates.

The price of Bangor (Pennsylvania) slates in Boston, at the same date, is very nearly the same as for Monson slates, except for 16 × 8's, which are $1 a square less.

Red slates cost from $12 to $12.50 per square.

PRICES OF MONSON (MAINE) SLATES.

SIZE.	Price per square.	SIZE.	Price per square.	SIZE.	Price per square.	SIZE.	Price per square.
24 × 14	$5 75	20 × 10	$6 50	16 × 9	$7 00	12 × 9	$5 75
24 × 12	6 00	18 × 12	6 25	16 × 8	7 50	12 × 8	6 00
22 × 14	5 75	18 × 11	6 50	14 × 12	6 00	12 × 7	5 50
22 × 12	6 00	18 × 10	6 75	14 × 10	6 50	12 × 6	5 00
22 × 11	6 00	18 × 9	6 50	14 × 9	6 50	11 × 8	5 50
20 × 14	6 00	16 × 12	6 25	14 × 8	6 75	11 × 7	5 00
20 × 12	6 25	16 × 11	6 50	14 × 7	6 50	10 × 8	5 00
20 × 11	6 25	16 × 10	7 00	12 × 10	5 75		

Shingles.

Tile = $\triangle^{9}_{12}$ = .37°

The average width of a shingle is four inches: hence, when shingles are laid four inches to the weather, each shingle averages sixteen square inches, and 900 are required for a square of roofing.

If 4½	inches	to the	weather,	800	will cover	a square.
5	"	"	"	720	"	"
5½	"	"	"	655	"	"
6	"	"	"	600	"	"

This is for common gable-roofs. In hip-roofs, where the shingles are cut more or less to fit the roof, add five per cent to above figures.

A carpenter will carry up and lay on the roof from fifteen hundred to two thousand shingles per day, or two squares to two squares and a half of plain gable-roofing.

One thousand shingles laid four inches to the weather will require five pounds of shingle-nails to fasten them on. Six pounds of fourpenny nails will lay one thousand split pine shingles.

Roofing-Tiles.

Spf. $26\frac{1}{2}$ - 30

Tiles are thin slabs of baked clay. They are extensively used in Europe for roofs, gutters, and house-siding, and, to some extent, in this country.

Plain roofing-tiles are usually made $\frac{5}{8}$ of an inch in thickness, $10\frac{1}{2}$ inches long, and $6\frac{1}{4}$ inches wide. They weigh from 2 to $2\frac{1}{2}$ pounds each, and expose about one-half to the weather. 740 tiles cover 100 superficial feet. They are hung upon the lath by two oak pins inserted into holes made by the moulder. Plain tiles are now made with grooves and fillets on the edges, so that they are laid without overlapping very far, the grooves leading the water. This is economical of tiles, and saves half of the weight, but is subject to leak in drifting rains, and to injury by hard frosts.

Pan-tiles, first used in Flanders, have a wavy surface, lapping under, and being overlapped by, the adjacent tiles of the same rank. They are made $14\frac{1}{2}$ by $10\frac{1}{2}$, expose ten inches to the weather, and weigh from 5 to $5\frac{1}{4}$ pounds each. 170 cover 100 square feet of surface.

Crown, ridge, hip, and valley tiles are semi-cylindrical, or segments of cylinders, used for the purposes indicated. A gutter-tile has been introduced in England, forming the lower course, being nailed to the lower sheathing-board or lath.

Siding-tiles are used as a substitute for weather-boarding. Holes are made in them when moulding, and they are secured to the lath by flat-headed nails. The gage, or exposed face, is sometimes indented to represent courses of brick. Fine mortar is introduced between them when they rest upon each other. Siding-tiles are sometimes called "weather-tiles" and "mathematical tiles." These names are derived from their exposure or markings. They are variously formed, having curved or crenated edges, and various ornaments, either raised or encaustic.

The unglazed tiles are inferior to slate, as they imbibe about one-seventh of their weight of water, and tend to rot the lath on which they are laid. Good roofing-slate only imbibes one two-hundredth part of its weight, and is nearly waterproof.

Tin Roofs.

[Revised for Fourth Edition.]

A tin roof of good material, properly put on, and kept properly painted, will last from thirty to forty years. It should not be painted for the first time until it has been well washed by rain, to get the grease off the tin; and all rosin, if used, should be carefully scraped off. One or more layers of felt-paper should be placed

under the tin, to serve as a cushion, and also to deaden the noise produced by the rain striking the tin.

For a steep roof, the tin should be put on with a standing groove, and with the cross seams double-locked and soldered. A very common and cheaper method for steep roof is, to double-lock both the vertical and cross seams, and fill the joint with white lead instead of soldering; but the other method is much the best. For flat roofs, the tin should be locked and soldered at all joints, and secured by tin cleats, and not by driving the nails through the tin itself.

In soldering the joints, rosin as a flux is generally preferred; although some roofers recommend the use of diluted chloride of zinc.

Roofing-plates are made of steel or iron, and covered with a mixture of lead and tin, and are designated as "tern," "leaded," or "roofing tin," in distinction from plates coated only with tin, and therefore called "bright tin." Roofing-plates are coated by two methods. The original manner of coating the plates was by dipping the black plate into the mixture of tin and lead, and allowing the sheets to absorb all the coating that was possible; and several brands of roofing-tin are still made by this process. The other process, by which the majority of roofing-plates are now made, is known as the "Patent-roller Process," by which the plates are put into a bath of tin and lead, and are passed through rolls, the pressure of which leaves on the iron or steel a thickness of coating which, to a great extent, determines the value of the plate. These rolls can be so adjusted as to leave a good amount of coating on the plate, an ordinary coating, or a very scant one; the heavier the coating, the more valuable the plate.

There have been only two sizes of roofing-plates made for a number of years; namely, 14 × 20 and 20 × 28: and of these two sizes, the larger is more generally used, from the fact that, being double the size of the smaller plate, it requires less seams on the roof, and consequently cheapens the cost of putting on.

Besides these two sizes, there is another size, 10 × 20, which is used for gutters and leader-pipe. A better roof will be obtained by using the 14 × 20 than the 20 × 28, because the seams are closer together, thus making the roof stronger; and, if put on with a standing seam, there is more allowance for expansion and contraction.

For steep roofs with standing groove, the tin should be laid with the smallest dimension for the width; as it makes the roof stronger, and allows a greater amount of expansion and contraction. Unfortunately, it is much cheaper to lay them the other way, as less

feet high exerts a pressure of about 0.86 of a pound, or just twice that exerted by a column one foot high. This pressure per square inch, due to head,[1] is irrespective of volume, or any thing else except vertical height of column. With these figures in mind, the calculation of the pressure per square inch due to any head is a simple matter. The following rules will be found valuable for reference :—

TO FIND PRESSURE IN POUNDS PER SQUARE INCH EXERTED BY A COLUMN OF WATER.—Multiply the height of the column, in feet, by 0.43.

TO FIND THE HEAD.—Multiply the pressure, in pounds per square inch, by 2.31.

Pressure of Water.—The weight of water or of other liquids is as the quantity, but the pressure exerted is as the vertical height.

Fluids press equally in all directions: hence any vessel or conduit containing a fluid sustains a pressure on the bottom equal to as many times the weight of the column of greatest height of that fluid as the area of the vessel is to the sectional area of the column.

Lateral Pressure.—The lateral pressure of a fluid on the sides of the vessel or conduit in which it is contained is equal to the product of the length multiplied by half the square of the depth and by the weight of the fluid in cubic unit of dimensions. The following formula is simple and satisfactory: multiply the submerged area in inches by the pressure due to one-half the depth. By "submerged area" is meant the surface upon which the water presses; for example, to find the lateral pressure upon the sides of a tank twelve feet long by twelve feet deep: 144 × 144 = 20736 inches of side. The pressure at the bottom will be 12 × 0.43 = 5.16 pounds, while the pressure at the top is 0, giving us, say, 2.6 pounds as the average: therefore 20736 × 2.6 = 53914 pounds.

Discharge of Water.—The quantity of water discharged during a given time from a given orifice, under different heads, is nearly as the square roots of the corresponding heights of the water in the reservoir or containing vessel above the surface of the orifice.

Small orifices, on account of friction, discharge proportionately less than those which are larger and of the same shape under the same pressure.

Circular apertures are the most efficacious, having less surface in proportion to area than any other form.

If a cylindrical horizontal tube through which water is discharged

[1] A head of water equals the height that the water rises above the orifice.

be of greater length than its diameter, the discharge is much increased. It can be lengthened with advantage to four times the diameter of the orifice.

TO FIND THE NUMBER OF UNITED-STATES GALLONS CONTAINED IN A FOOT OF PIPE OF ANY DIAMETER. — Square the diameter of the pipe in inches, and multiply the square by 0.0408.

Velocity of Flow of Water. — Water which has a chance to flow downward does so with a velocity in exact proportion to its head. The following table gives the velocity of flow of water due to heads of from one to forty feet: —

Velocity in Feet per Second due to Heads of from 1 to 40 Feet.[1]

Head.	Velocity.	Head.	Velocity.	Head.	Velocity.	Head.	Velocity.
0.5	5.67	10.5	25.98	20.5	36.31	30.5	44.29
1.0	8.02	11.0	26.60	21.0	36.75	31.0	44.65
1.5	9.82	11.5	27.19	21.5	37.18	31.5	45.01
2.0	11.34	12.0	27.78	22.0	37.61	32.0	45.37
2.5	12.68	12.5	28.35	22.5	38.04	32.5	45.72
3.0	13.89	13.0	28.91	23.0	38.46	33.0	46.07
3.5	15.00	13.5	29.46	23.5	38.88	33.5	46.42
4.0	16.04	14.0	30.00	24.0	39.29	34.0	46.76
4.5	17.01	14.5	30.54	24.5	39.69	34.5	47.10
5.0	17.93	15.0	31.06	25.0	40.10	35.0	47.44
5.5	18.81	15.5	31.57	25.5	40.50	35.5	47.78
6.0	19.64	16.0	32.08	26.0	40.89	36.0	48.12
6.5	20.44	16.5	32.58	26.5	41.28	36.5	48.45
7.0	21.22	17.0	33.06	27.0	41.67	37.0	48.78
7.5	21.96	17.5	33.55	27.5	42.05	37.5	49.11
8.0	22.68	18.0	34.02	28.0	42.44	38.0	49.44
8.5	23.38	18.5	34.49	28.5	42.81	38.5	49.76
9.0	24.06	19.0	34.96	29.0	43.19	39.0	50.08
9.5	24.72	19.5	35.41	29.5	43.56	39.5	50.40
10.0	25.36	20.0	35.86	30.0	43.92	40.0	50.72

In plumbing-work we cannot secure this velocity in the flow of water through pipes, because of the friction which constantly tends to diminish it. The longer the pipe, the greater the friction and consequent retardation of the flow. In the following table we have the head of water consumed by friction in pipes one yard long and from one to four inches in diameter. This table shows the head of water required to produce a given flow per minute. By means of the rules given on p. 538 it is made applicable to any length of pipe; and a variety of problems relating to lengths and diameters of pipe, discharge in gallons, and head in feet, are solved by it.

[1] Box's Hydraulics.

Head of Water consumed by Friction in Pipes one Yard Long.[1]

Gallons per minute.	Diameter of the Pipe, in Inches.						
	1	1½	2	2¼	3	3½	4
	Head of Water, in Feet.						
1	0.0041	0.00054	0.00012	0.000042	0.000016	0.0000078	0.000004
2	0.0164	0.00216	0.00051	0.000168	0.000067	0.0000313	0.000016
3	0.0370	0.00487	0.00115	0.000379	0.000152	0.0000705	0.000036
4	0.0658	0.00867	0.00205	0.000674	0.000271	0.0001250	0.000064
5	0.1028	0.01354	0.00321	0.001053	0.000423	0.000195	0.000100
6	0.1481	0.01950	0.00463	0.001517	0.000609	0.000282	0.000144
7	0.2016	0.02655	0.00630	0.002064	0.000830	0.000383	0.000196
8	0.2633	0.03468	0.00823	0.002696	0.001084	0.000501	0.000257
9	0.3333	0.04389	0.01041	0.003413	0.001372	0.000634	0.000325
10	0.4110	0.05410	0.01286	0.004210	0.001690	0.000783	0.000401
20	1.64	0.21670	0.05140	0.016850	0.006770	0.00313	0.001600
30	3.70	0.48770	0.115	0.037920	0.0152	0.00707	0.003610
40	6.58	0.86700	0.205	0.067420	0.0271	0.01253	0.006430
50	10.28	1.35	0.321	0.1053	0.0423	0.01958	0.010040
60	14.81	1.95	0.463	0.1517	0.0609	0.02820	0.014450
70	20.16	2.65	0.630	0.2064	0.0830	0.03839	0.019690
80	26.33	3.46	0.823	0.2696	0.1084	0.05014	0.025720
90	33.33	4.38	1.041	0.3413	0.1372	0.06346	0.032570
100	41.1	5.4	1.28	0.421	0.169	0.078	0.0401
110	49.7	6.5	1.55	0.509	0.205	0.094	0.0486
120	59.2	7.8	1.85	0.606	0.243	0.112	0.0578
130	69.5	9.1	2.17	0.712	0.286	0.132	0.0679
140	80.6	10.6	2.52	0.825	0.332	0.153	0.0788
150	92.5	12.1	2.89	0.948	0.381	0.176	0.0904
160	105.3	13.8	3.29	1.078	0.433	0.200	0.1028
170	118.9	15.6	3.71	1.217	0.485	0.226	0.1161
180	133.3	17.5	4.16	1.365	0.549	0.253	0.1312
190	148.5	19.5	4.64	1.521	0.611	0.282	0.1450
200	164.6	21.6	5.14	1.685	0.677	0.313	0.1607
210	181.4	23.8	5.67	1.858	0.747	0.345	0.1772
220	199.1	26.2	6.22	2.039	0.819	0.379	0.1945
230	217.6	28.6	6.80	2.229	0.896	0.414	0.2126
240	237.0	31.2	7.40	2.427	0.975	0.451	0.2314
250	257.1	33.8	8.03	2.633	1.058	0.489	0.2511
260	278.1	36.6	8.69	2.848	1.145	0.529	0.2716
270	[illegible]	39.5	9.37	3.071	1.234	0.571	0.2929
280	[illegible]	42.4	10.08	3.303	1.328	0.614	0.3150
290	[illegible]	45.5	10.81	3.541	1.424	0.658	0.3379
300	370.5	48.7	11.58	3.792	1.524	0.705	0.3617
310	[illegible]	52.0	12.35	4.050	1.627	0.752	0.3862
320	421.3	55.5	13.16	4.215	1.734	0.802	0.4115
330	[illegible]	59.0	14.00	4.580	1.844	0.853	0.4376
340	[illegible]	62.6	14.87	4.871	1.958	0.905	0.4645
350	[illegible]	66.3	15.75	5.162	2.075	0.959	0.4923
360	[illegible]	70.2	16.66	5.461	2.196	1.015	0.5248
370	[illegible]	74.1	17.60	5.760	2.326	1.072	0.5502
380	[illegible]	78.2	18.57	6.085	2.446	1.131	0.5803
390	625.8	82.4	19.56	6.408	2.576	1.191	0.6112
400	[illegible]	86.7	20.57	6.742	2.710	1.253	0.6430
410	[illegible]	91.0	21.61	7.083	2.847	1.317	0.6755
420	[illegible]	95.5	22.68	7.433	2.988	1.382	0.7089
430	[illegible]	100.1	23.80	7.790	3.130	1.448	0.7430
440	[illegible]	104.9	24.80	8.150	3.270	1.516	0.7780
450	[illegible]	109.7	25.90	8.530	3.430	1.586	0.8130
460	[illegible]	114.6	27.10	8.910	3.580	1.657	0.8500
470	[illegible]	119.7	28.40	9.300	3.740	1.730	0.8870
480	[illegible]	124.8	29.60	9.700	3.900	1.805	0.9250
490	[illegible]	130.1	30.80	10.110	4.060	1.881	0.9640
500	[illegible]	135.4	32.10	10.530	4.230	1.958	1.0040

[1] Box's Hydraulics.

The practical application of this table will be found in the following rules:—

TO FIND THE HEAD OF WATER, WHEN DIAMETER AND LENGTH OF PIPE, AND NUMBER OF GALLONS DISCHARGED PER MINUTE, ARE KNOWN.—In the above table the head due to a length of one yard is found opposite the number of gallons. Multiply that number by the given length in yards, and we have the required head in feet. Thus, to find the head necessary to deliver 130 gallons per minute by a pipe 4 inches in diameter, 500 yards long: opposite 130 gallons in the table, and under 4 inches in diameter, is 0.679, which, multiplied by 500, gives 339.5 feet, the head sought.

TO FIND THE DIAMETER OF THE PIPE, WHEN HEAD, LENGTH OF PIPE, AND THE NUMBER OF GALLONS DISCHARGED PER MINUTE, ARE KNOWN.—Divide the head of water in feet by the length of the pipe in yards, and the number nearest to this in the table opposite the number of gallons will be found under the required diameter.

TO FIND THE NUMBER OF GALLONS DISCHARGED, WHEN THE HEAD, LENGTH OF PIPE AND ITS DIAMETER, ARE KNOWN.—Divide the head of water in feet by the given length in yards, and the nearest number thereto in the table under the diameter will be found opposite the required number of gallons.

TO FIND THE LENGTH, WHEN THE HEAD, NUMBER OF GALLONS PER MINUTE, AND DIAMETER OF PIPE, ARE KNOWN.—Divide the given head by the head for one yard, found in the table under the given diameter and opposite the given number of gallons, and the result is the required length.

The actual discharge of pipes is easily calculated with approximate accuracy by Prony's formula. In using this formula, find the discharge in gallons per minute by multiplying the head in inches by the diameter of the pipe in inches, and divide the product by the length of the pipe in inches $\left(\frac{H \times d}{L}\right)$. In the following table, find the number nearest to the quotient thus obtained in the first column, and the discharge in gallons per minute will be found opposite it, under the diameter of the pipe used.

The discharge of small pipes may be calculated with sufficient accuracy for practical purposes from the following convenient table, showing the quantity of water that will flow through a pipe 500 feet long in 24 hours, with a pressure due to a head of ten feet:—

$\frac{3}{8}$-inch bore . . .	576 gallons.	$\frac{3}{4}$-inch bore . .	3,200 gallons.
$\frac{1}{2}$-inch " . . .	1,150 "	1-inch " . .	6,624 "
$\frac{5}{8}$-inch " . . .	2,040 "	$1\frac{1}{4}$-inch " . .	10,000 "

This often bursts pipes which are amply strong to resist a great deal more l pressure to to increase the

The [redacted] gives the relation of size and [redacted] [redacted] pipes. These figures are compiled from the results of careful tests.

Weight and Strength of Lead Pipes.

Caliber.	Mark	Weight per foot.	Thickness.	Mean bursting pressure.	Safe working pressure.	Caliber.	Mark.	Weight per foot	Thickness.	Mean bursting pressure.	Safe working pressure.
ins		lb. oz.	ins.	lbs	lbs.	ins.		lb. oz.	ins.	lbs.	lbs.
3/8	AAA	1 12	0.18	1968	492	1	A	4 0	0.21	857	214
3/8	AA	1 6	0.15	1627	406	1	B	3 4	0.17	745	186
3/8	A	1 2	0.13	1381	347	1	C	2 8	0.14	562	140
3/8	B	1 0	0.125	1342	335	1	D	2 4	0.125	518	129
3/8	C	0 14	0.11	1187	296	1	E	2 0	0.10	475	118
3/8	-	0 10	0.087	1085	271	1	-	1 8	0.09	325	81
7/16	-	0 9¼	0.08	775	193	1¼	AAA	6 12	0.275	962	240
1/2	AAA	3 0	0.25	1787	446	1¼	AA	5 12	0.25	823	205
1/2	-	2 8	0.225	1655	413	1¼	A	4 11	0.21	685	171
1/2	AA	2 0	0.18	1393	346	1¼	B	3 11	0.17	546	136
1/2	A	1 10	0.16	1285	321	1¼	C	3 0	0.135	420	105
1/2	B	1 3	0.125	980	245	1¼	D	2 8	0.125	350	87
1/2	C	1 0	0.10	782	195	1¼	-	2 0	0.095	322	80
[illegible]	D	0 9	0.065	468	117	1½	AAA	8 0	0.29	742	185
1/2	-	0 10	0.07	556	139	1½	AA	7 0	0.25	700	175
1/2	-	0 12	0.0[illegible]	625	156	1½	A	6 4	0.22	628	157
5/8	AAA	3 8	0.23	1548	387	1½	B	5 0	0.18	506	126
5/8	AA	2 12	0.21	1380	345	1½	C	4 4	0.15	430	107
5/8	A	2 8	0.18	1152	288	1½	D	3 8	0.14	315	78
5/8	B	2 0	0.16	987	246	1½	-	3 0	0.12	245	61
5/8	C	1 7	0.117	795	198	1¾	B	5 0	-	-	116
5/8	D	1 4	0.10	708	177	1¾	C	4 0	-	-	93
3/4	AAA	4 14	0.20	1462	365	1¾	D	3 10	0.125	318	70
3/4	AA	3 8	0.225	1225	306	2	AAA	10 11	0.30	611	152
3/4	A	3 0	0.19	1072	268	2	AA	8 14	0.25	511	127
3/4	B	2 3	0.15	865	216	2	A	7 0	0.21	405	101
3/4	C	1 12	0.125	782	195	2	B	6 0	0.19	360	90
3/4	D	1 3	0.09	505	126	2	C	5 0	0.16	260	65
1	AAA	6 0	0.30	1230	307	2	D	4 0	0.09	200	50
1	AA	4 8	0.23	910	227						

Discharge of Pipes by Prony's Formula.

$\frac{H \times d}{L}$	Velocity in feet per second.	Diameter of the Pipe, in Inches. 1	$1\frac{1}{2}$	2	$2\frac{1}{2}$	3	$3\frac{1}{2}$	4	5	6
		Gallons discharged per Minute.								
0.00002402	0.025	0.0511	0.1150	0.2045	0.3196	0.4602	0.626	0.818	1.278	1.841
0.00005437	0.05	0.1022	0.2301	0.4091	0.6392	0.9204	1.252	1.636	2.556	3.682
0.0000908	0.075	0.1534	0.3450	0.6136	0.9588	1.381	1.878	2.454	3.834	5.523
0.0001341	0.100	0.2045	0.4602	0.8182	1.278	1.841	2.504	3.273	5.113	7.363
0.0001836	0.125	0.2556	0.5750	1.023	1.598	2.301	3.130	4.090	6.390	9.205
0.0002394	0.15	0.3067	0.6900	1.227	1.917	2.761	3.756	4.908	7.668	11.05
0.0003016	0.175	0.3578	0.8053	1.432	2.237	3.221	4.382	5.728	8.947	12.83
0.0003702	0.2	0.4090	0.9204	1.636	2.557	3.682	5.008	6.546	10.23	14.73
0.0004452	0.225	0.4601	1.035	1.841	2.876	4.142	5.634	7.363	11.50	16.57
0.0005266	0.25	0.5112	1.150	2.045	3.196	4.602	6.260	8.160	12.78	18.41
0.0006140	0.275	0.5624	1.265	2.250	3.515	5.062	6.886	9.000	14.06	20.25
0.0007080	0.3	0.6135	1.381	2.454	3.835	5.522	7.512	9.819	15.34	22.00
0.0008087	0.325	0.6646	1.496	2.659	4.154	5.982	8.138	10.64	16.62	23.93
0.0009154	0.35	0.7157	1.611	2.864	4.474	6.443	8.764	11.46	17.89	25.77
0.0010286	0.375	0.7669	1.726	3.068	4.794	6.903	9.390	12.27	19.17	27.61
0.0011480	0.4	0.8180	1.841	3.273	5.113	7.363	10.02	13.09	20.45	29.45
0.001274	0.425	0.8691	1.955	3.477	5.433	7.823	10.64	13.91	21.73	31.29
0.001406	0.45	0.9202	2.071	3.682	5.757	8.284	11.27	14.73	23.01	33.13
0.001545	0.475	0.9713	2.186	3.886	6.077	8.744	11.89	15.55	24.29	34.97
0.001690	0.5	1.023	2.301	4.091	6.392	9.204	12.52	16.37	25.57	36.82
0.002	0.55	1.125	2.531	4.500	7.031	10.12	13.77	18.00	28.12	40.50
0.00233	0.6	1.227	2.761	4.909	7.670	11.04	15.02	19.64	30.68	44.18
0.002693	0.65	1.329	2.991	5.318	8.309	11.96	16.28	21.27	33.23	47.86
0.003079	0.7	1.431	3.221	5.727	8.948	12.88	17.53	22.91	35.79	51.54
0.003490	0.75	1.533	3.450	6.136	9.588	13.81	18.78	24.54	38.34	55.23
0.003926	0.8	1.636	3.682	6.544	10.23	14.73	20.03	26.18	40.90	58.90
0.004388	0.85	1.738	3.912	6.954	10.86	15.65	21.29	27.82	43.46	62.59
0.004876	0.9	1.841	4.142	7.363	11.51	16.57	22.53	29.46	46.02	66.27
0.005928	1.0	2.045	4.602	8.182	12.78	18.41	25.04	32.73	51.13	73.63
0.00648	1.05	2.147	4.832	8.591	13.42	19.33	26.29	34.37	53.69	77.31
0.00708	1.1	2.249	5.062	9.000	14.06	20.25	27.54	36.00	56.24	80.99
0.007691	1.15	2.351	5.292	9.409	14.70	21.15	28.80	37.64	58.80	84.67
0.008338	1.2	2.454	5.522	9.818	15.34	22.09	30.05	39.28	61.36	88.36
0.009	1.25	2.556	5.753	10.23	15.96	23.01	31.30	40.91	63.91	92.04

Having determined the pressure due to head with which he has to deal, and the size of the pipe needed to discharge a given quantity in a given time, the plumber must calculate the strength which his pipe must possess to resist this pressure under all conditions. This he need not do with absolute accuracy, for the reason that he must use the pipe he finds in the market; but the strength of the sizes in the market is known, and on the basis of this knowledge he can determine the weight of pipe he requires. In all such calculations, however, there should be a liberal margin for safety. The pipe may corrode, external influences may weaken it, and extraordinary pressures may be brought to bear upon it, — as by the sudden closing of a cock, which, owing to the incompressible nature of water, causes it to strike a powerful blow, due to the suddenly arrested momentum of the entire column of water in the pipe.

This often bursts pipes which are amply strong to resist a great deal more pressure to which they are subjected. Other to increase the pressure, and tax the resisti the strong enough to bear these ing. The the relation of size and pipes. These figures are compiled from the results of careful tests.

Weight and Strength of Lead Pipes.

Caliber.	Mark.	Weight per foot.	Thickness.	Mean bursting pressure	Safe working pressure	Caliber.	Mark.	Weight per foot.	Thickness.	Mean bursting pressure.	Safe working pressure.
ins.		lb. oz.	ins.	lbs.	lbs.	ins.		lb. oz.	ins.	lbs.	lbs.
3/8	AAA	1 12	0.18	1968	492	1	A	4 0	0.21	857	214
3/8	AA	1 6	0.15	1627	406	1	B	3 4	0.17	745	186
3/8	A	1 2	0.13	1381	347	1	C	2 8	0.14	562	140
3/8	B	1 0	0.125	1342	335	1	D	2 4	0.125	518	129
3/8	C	0 14	0.11	1187	296	1	E	2 0	0.10	475	118
3/8	–	0 10	0.087	1085	271	1	–	1 8	0.09	325	81
7/16	–	0 9½	0.08	775	193	1¼	AAA	6 12	0.275	962	240
1/2	AAA	3 0	0.25	1787	446	1¼	AA	5 12	0.25	823	205
1/2	–	2 8	0.225	1655	413	1¼	A	4 11	0.21	685	171
1/2	AA	2 0	0.18	1393	348	1¼	B	3 11	0.17	546	136
1/2	A	1 10	0.16	1285	321	1¼	C	3 0	0.135	420	105
1/2	B	1 3	0.125	980	245	1¼	D	2 8	0.125	350	87
1/2	C	1 0	0.10	782	195	1¼	–	2 0	0.095	322	80
1/2	D	0 9	0.065	468	117	1½	AAA	8 0	0.29	742	185
1/2	–	0 10	0.07	550	139	1½	AA	7 0	0.25	700	175
1/2	–	0 12	0.0[illegible]	625	156	1½	A	6 4	0.22	628	157
5/8	AAA	3 8	0.23	1548	387	1½	B	5 0	0.18	506	126
5/8	AA	2 12	0.21	1380	345	1½	C	4 4	0.15	430	107
5/8	A	2 8	0.18	1152	288	1½	D	3 8	0.14	315	78
5/8	B	2 0	0.16	987	246	1½	–	3 0	0.12	245	61
5/8	C	1 7	0.117	795	198	1¾	B	5 0	–	–	116
5/8	D	1 4	0.10	708	177	1¾	C	4 0	–	–	93
3/4	AAA	4 14	0.20	1462	365	1¾	D	3 10	0.125	318	79
3/4	AA	3 8	0.225	1225	306	2	AAA	10 11	0.30	611	152
3/4	A	3 0	0.19	1072	265	2	AA	8 14	0.25	511	127
3/4	B	2 3	0.15	865	216	2	A	7 0	0.21	405	101
3/4	C	1 12	0.125	782	194	2	B	6 0	0.19	360	90
3/4	D	1 3	0.09	505	126	2	C	5 0	0.16	260	65
1	AAA	6 0	0.30	1230	307	2	D	4 0	0.09	200	50
1	AA	4 8	0.25	910	227						

Wrought-iron pipes suitable for water service range in diameter from half an inch to sixteen inches. The tables on pp. 621, 622, show the weight of the various sizes manufactured.

Messrs. Tasker & Co., of the Pascal Iron-Works, Philadelphia, subject the pipes which they manufacture to the following tests:—

One-half to one and one-fourth inch, butt-welded, 300 pounds hydraulic pressure per square inch.

One and one-half to ten inch, lap-welded, 500 pounds hydraulic pressure per square inch.

Practically they are strong enough to bear any pressure with which the plumber has to deal. The same is true of drawn brass and copper pipes.

The pressures to be dealt with in American plumbing practice vary through a wide range. In cities supplied by what are known as gravity-works — i.e., where dependence is placed on natural head at the distributing reservoir, as in New York — the pressure of water is often very light.

Where pumping machinery is used, and a high head is maintained in tall stand-pipes, or the pumps deliver directly into the mains, we sometimes get pressures of one hundred pounds to the square inch, and upward.

MEMORANDA FOR PAINTERS.

[From "Builders' Guide and Price Book."]

Painting.

Painters' work is generally estimated by the yard, and the cost depends upon the number of coats applied, besides the quality of the work, and the material to be painted.

One coat, or *priming*, will take, for 100 yards of painting, 20 pounds of lead and 4 gallons of oil. Two-coat work, 40 pounds of lead and 4 gallons of oil. Three-coat, the same quantity as two coats; so that a fair estimate for 100 yards of three-coat work would be 100 pounds of lead and 16 gallons of oil.

1	gallon	priming color	will cover	50	superficial	yards.
1	"	white zinc	"	50	"	"
1	"	white paint	"	44	"	"
1	"	lead color	"	50	"	"
1	"	black paint	"	50	"	"
1	"	stone color	"	44	"	"

1 gallon	yellow paint	will cover	44	superficial	yards.
1 "	blue color	"	45	"	"
1 "	green paint	"	45	"	"
1 "	bright emerald green	"	25	"	"
1 "	bronze green	"	75	"	"

One pound of paint will cover about 4 superficial yards the first coat, and about 6 each additional coat. One pound of putty, for stopping, every 20 yards. One gallon of tar and 1 pound of pitch will cover 12 yards superficial the first coat, and 17 yards each additional coat.

A square yard of new brick wall requires, for the first coat of paint in oil, ¾ of a pound; and for the second, 3 pounds; and for the third, 4 pounds.

A day's work on the outside of a building is 100 yards of first coat, and 80 yards of either second or third coat. An ordinary door, including casings, will, on both sides, make 8 to 10 yards of painting, or about 5 yards to a door without the casings. An ordinary window makes about 2½ or 3 yards.

Fifty yards of common graining is a day's work for a grainer and one man to rub in. In painting blinds of ordinary size, 12 is a fair day's work for one coat, and 9 pounds of lead and 1 gallon of oil will paint them.

LIGHTNING CONDUCTORS.

Rules for the erection of lightning conductors, issued in 1882 by the Explosive Department of the Home Office to the occupiers of all factories and magazines for explosives, and to those local and police authorities upon whom devolves the inspection of stores of explosives:

1. *Material of Rod.*—Copper, weighing not less than 6 oz. per foot run, the electrical conductivity of which is not less than 90 per cent. of that of pure copper, either in the form of rod, tape, or rope of stout wires, no individual wire being less than No. 12 B. W. G. (.109 inch). Iron may be used, but should not weigh less than 2¼ pounds per foot run.

2. *Joints.*—Every joint, besides being well cleaned and screwed, scarfed, or riveted, should be thoroughly soldered.

3. *Form of Points.*—The point of the upper terminal[1] of the

[1] The upper terminal is that portion of the conductor which is between the top of the edifice and the point of the conductor.

conductor should not have a sharper angle than 90°. A foot below the extreme point a copper ring should be screwed and soldered on to the upper terminal, in which ring should be fixed three or four sharp copper points, each about six inches long. It is desirable that these points should be so platinized, gilded or nickel-plated, as to resist oxidation.

4. *Number and Height of Upper Terminals.*—The number of conductors or upper terminals required will depend upon the size of the building, the material of which it is constructed, and the comparative height above ground of the several parts. No general rule can be given for this, except that it may be assumed that the space protected by the conductor is, as a rule, a cone, the radius of whose base is equal to the height of the conductor from the ground.

5. *Curvature.* The rod should not be bent abruptly round sharp corners. In no case should the length of a curve be more than half as long again as its chord. A hole should be drilled in string-courses or other projecting masonry, when possible, to allow the rod to pass freely through it.

6. *Insulators.*—The conductor should not be kept from the building by glass or other insulators, but attached to it by fastenings of the same metal as the conductor itself is composed of.

7. *Fixing.* Conductors should preferentially be taken down the side of the building which is most exposed to rain. They should be held firmly, but the holdfasts should not be driven in so tightly as to pinch the conductor or prevent contraction and expansion due to change of temperature.

8. *Other Metal Work.*—All metallic spouts, gutter, iron doors, and other masses of metal about the building should be electrically connected with the conductor.

9. *Earth Connection.* It is most desirable that, whenever possible, the lower extremity of the conductor should be buried in permanently damp soil. Hence, proximity to rain water pipes and to drains or other water is desirable. It is a very good plan to bifurcate the conductor close below the surface of the ground, and to adopt two of the following methods for securing the escape of the lightning into the earth: (1) A strip of copper tape may be led from the bottom of the rod to a gas or water main (not merely to a leaden pipe), if such exist near enough, and be soldered to it; (2) a tape may be soldered to a sheet of copper, 3 feet × 3 feet × [illegible] inch thick, buried in permanently wet earth and surrounded by cinders or coke; (3) many yards of copper tape may be laid in a trench filled with coke, having not less than 18 square feet of copper exposed.

10. *Protection from Theft, etc.*—In places where there is any likelihood of the copper being stolen or injured, it should be protected by being enclosed in an iron gas-pipe, reaching ten feet (if there is room) above ground and some distance into the ground.

11. *Painting.*—Iron conductors, galvanized or not, should be painted. It is optional with copper ones.

12. *Inspection.*—When the conductor is finally fixed it should in all cases be examined and tested by a qualified person, and this should be done in the case of new buildings after all work on them is finished.

Periodical examination and testing, should opportunities offer, are also very desirable, especially when iron earth connections are employed.

SIMPLE ELECTRICAL DEFINITIONS AND FORMULÆ.

[From "Modern Light and Heat."]

The *Volt* is the unit of electro-motive force, which in formulæ is symbolized by E.

Electro-motive force, which is the force that moves electricity, is usually written E. M. F., and various writers use it to express potential, difference of potential, electric pressure, and electric force.

Potential and E M. F. are different ways of regarding the same agency and are equal in value. Both are measured in volts, and are equal at the same point. Potential relates to the inductive circuit, and E. M. F. relates to the conductive circuit.

One volt will force one ampère of current through one ohm of resistance. Its value is purely arbitrary, but fixed.

The *Ohm* is the unit of resistance, which in formulæ is symbolized by R.

Its value is not absolutely known, but all electricians in 1886 agreed to consider it, for 10 years, as equal to the resistance of a column of pure mercury 1 square millimeter in section and 106 centimeters long at the temperature of melting ice. A copper wire 95 per cent conductivity, $\frac{1}{1000}$ of an inch in diameter, and 10 feet long has about 100 ohms resistance.

One ohm is that resistance through which one ampère of current will flow at a pressure of one volt of E. M. F.

The *Ampère* is the unit of current per second, which in formulæ is symbolized by C. Its value may be defined as that quantity of

electricity which flows per second through one ohm of resistance, when impelled by one volt of E. M F.

One ampère of current flowing through a bath will deposit 0.017253 grain of silver, or 0.005084 grain of copper per second.

The relations which exist between E. M. F., resistance, and current are known as Ohm's Law. Its simplest expressions are as follows :

In an electrical circuit the *Current* in ampères may be found by dividing the E. M. F. in volts by the resistance in ohms.

The E. M. F. in volts may be found by multiplying the current in ampères by the resistance in ohms.

The *Resistance* in ohms may be found by dividing the E. M. F. in volts by the current in ampères.

In a given resistance an increase of E. M. F. must be accompanied by a proportional increase of current ; or an increase of current must be accompanied by a proportional increase of E. M. F.; but an increase of resistance will be accompanied by a proportional increase of E. M. F., or a proportional decrease of current ; and a decrease of resistance will be accompanied by a proportional decrease of E. M. F., or a proportional increase of current.

According to these relations, it is seen that C and R are each the reciprocal of the other multiplied by E ; that is to say, that C and R limit and define each other where E is a fixed quantity.

In a given resistance, *Energy*, such as work or heat, varies as the square of the current or of the electro-motive force ; that is, by doubling the E. M. F. the energy becomes four times as great ; by trebling the E. M. F. the energy is nine times as great.

Power is the rate of doing work, and is proportional to the E. M. F. multiplied by the current.

The *Watt* is the unit of electrical power.

One volt multiplied by one ampère equals one watt.

One *Electrical Horse-power* equals 746 watts. That is to say, a current of 1 ampère and 746 volts would be one electrical horse-power. And one horse power expended wholly in producing electric energy would generate 1 ampère in 746 ohms resistance, or 746 ampères in 1 ohm resistance.

In reading French text-books it must be remembered that *cheval-vapeur*, or French horse-power, equals only 736 watts.

For copper wire the square of diameter, with the following constants, will give the following properties, *d* being equal to diameter :

Feet per pound, divide	330560	by d^2
Yards per pound, divide	110187	by d^2

Grains per foot, multiply	0 0211761	by d^2
Pounds per 1,000 feet, multiply	0.0030252	by d^2
Pounds per mile, multiply	0.015973	by d^2
Pounds per nautical mile, multiply	0.018414	by d^2

The same constants used in the opposite manner will give the area, from which the diameter may be found by dividing it by .7854 and extracting the square root.

The following figures furnish useful data as to copper wire; they are the resistances of a wire $\frac{1}{1000}$ of an inch diameter, and of the length named, at 60° Fahrenheit. Divided by the sectional area they will give:

10.3365	will give ohms per foot.
31.0095	will give ohms per yard.
54577.	will give ohms per mile.
62918.	will give ohms per nautical mile.
3416825.	divided by d^4 will give ohms per pound.
0.0967447	multiplied by d^2 will give feet per ohm

As a copper wire becomes warm, so does its resistance increase. Between the freezing and boiling points of water this ratio is very nearly fixed. For practical purposes the resistance of a copper wire may be said to increase .215 of 1 per cent. for every degree Fahrenheit.

It is convenient to remember that the weight of a wire is directly, and the resistance inversely, proportional to the square of its diameter, and that the resistance of a wire varies inversely as the section, and, therefore, inversely as the square of the diameter, and also inversely as to the weight of a given length. It also varies directly as to the length of a given weight.

TABLE SHOWING DIFFERENCE IN WEIGHT OF COPPER WIRE.

No.	B. & S., or American Gauge. Lbs. per 1,000 feet.	Birmingham Gauge. Lbs. per 1,000 feet.	New British Standard Gauge. Lbs. per 1,000 feet.
4–0	639.33	623.925	484.03
3–0	507.01	546.76	418.63
2–0	402.09	437.107	366 36
0	319.04	349.928	317.54
1	252.88	272.435	272.27
2	200.54	244.15	230.44
3	159.03	202.964	192.11
4	126.12	171.465	162.88
5	100 01	146.51	135.96
6	79.32	124.742	111 52
7	62.90	98.076	93.71
8	49.88	82.41	77.445
9	39.56	66.305	62.730
10	31.37	54.354	49.565
11	24.88	43.59	40.707
12	19.73	35.964	32.720
13	15.65	27.319	25.605
14	12.41	20.853	19.361
15	9.84	15.692	15.683
16	7.81	12.789	12.391
17	6.19	10.18	9.4869
18	4.91	7.268	6.9700
19	3.78	5 340	4.8403
20	3.09	3.708	3.9206
21	2.45	3.099	3.0978
22	1.94	2.373	2.3708
23	1.54	1.892	1.7425
24	1.22	1.465	1.4642
25	.97	1.211	1.2100
26	.77	.9807	.98015
27	.61	.7749	.81365
28	.48	.5933	.66263
29	.38	.5116	.55953
30	.30	.4359	.46515
31	.24	.3027	.40707
32	.19	.2452	.35286
33	.15	.1937	.30253
34	.12	.1483	.25605
35	.10	.07568	.21346
36	.08	.04843	.17473

TABLES OF DIFFERENT GAUGES, WITH THEIR RESPECTIVE DIAMETERS AND AREAS.

RESISTANCE OF PURE COPPER AT 75° FAHRENHEIT.

No.	Ohms per 1,000 feet.	Feet per ohm.	Ohms per pound.
0000	.051	19,605.69	.0000798
000	.064	15,547.87	.000127
00	.081	12,330.36	.000202
0	.102	9,783.63	.000320
1	.129	7,754.66	.00051
2	.163	6,149.78	.000811
3	.205	4,876.73	.001289
4	.259	3,867.62	.00205
5	.326	3,067.06	.00326
6	.411	2,432.22	.00518
7	.519	1,928.75	.00824
8	.652	1,529 69	.01311
9	.824	1,213 22	.02083
10	1.040	961.91	.03314
11	1 311	762.93	.05269
12	1.653	605 03	.08377
13	2 084	479.80	.13321
14	2.628	380.51	.2118
15	3.314	301.75	.3368
16	4.179	239.32	.5355
17	5.269	189.78	.8515
18	6.645	150.50	1.3539
19	8.617	116.05	2.2772
20	10.566	94.65	3.423
21	13.323	75.06	5.443
22	16.799	59.53	8.654
23	21.185	47.20	13.763
24	26.713	37.43	21.885
25	33.684	29.69	34.795
26	42.477	23.54	55.331

EQUIVALENT OF 32DS OF AN INCH IN THOUSANDTHS OF AN INCH.

1–32	equals		.03125	17–32	equals		.53125
2	"	"	.06250	18	"	"	.5625
3	"	"	.09375	19	"	"	.59375
4	"	"	.125	20	"	"	.625
5	"	"	.15625	21	"	"	.65625
6	"	"	.18750	22	"	"	.68750
7	"	"	.21865	23	"	"	.71875
8	"	"	.250	24	"	"	.750
9	"	"	.28125	25	"	"	.78125
10	"	"	.31250	26	"	"	.81250
11	"	"	.34375	27	"	"	.84375
12	"	"	.375	28	"	"	.875
13	"	"	.40625	29	"	"	.90625
14	"	"	.43750	30	"	"	.93750
15	"	"	.46875	31	"	"	.96875
16	"	"	.500	32	"	"	1.000

RULES AND REQUIREMENTS OF THE NATIONAL AND NEW YORK BOARD OF FIRE UNDERWRITERS FOR THE INSTALLATION OF ELECTRIC LIGHT AND POWER.

AS RECOMMENDED BY

THE UNDERWRITERS' INTERNATIONAL ELECTRIC ASSOCIATION, JANUARY, 1894.

The use of wire-ways for rendering concealed wiring permanently accessible, is most heartily indorsed and recommended; and this method of accessible concealed construction is advised for general use.

Architects are urged, when drawing plans and specifications, to make provision for the channelling and pocketing of buildings for electric light or power wires, and in specifications for electric gas lighting to require a two-wire circuit, whether the building is to be wired for electric lighting or not, so that no part of the gas fixtures or gas piping be allowed to be used for the gas-lighting circuit.

CENTRAL STATIONS.—CLASS A.

For Light or Power.

The rules under this class, not being of special interest to architects, are omitted.

CLASS B.—ARC (SERIES) SYSTEMS.

Over 300 *Volts.*

10. **Outside Conductors.**—All outside, overhead conductors (including services):

(*a*) Must be covered with some *approved* insulating material, not easily abraded, firmly secured to properly insulated and substantially built supports, all tie wires having an insulation equal to that of the conductors they confine.

(*b*) Must be so placed that moisture cannot form a cross connection between them, not less than a foot apart, and not in contact with any substance other than their insulating supports.

(*c*) Must be at least seven feet above the highest point of flat roofs, and at least one foot above the ridge of pitched roofs over which they pass or to which they are attached.

(*d*) Must be protected by *dead insulated guard irons* or *wires* from possibility of contact with other conducting wires or substances to which current may leak. Special precautions of this kind must be taken where sharp angles occur or where any wires might possibly come in contact with electric light or power wires.

(*e*) Must be provided with petticoat insulators of glass or porcelain. Porcelain knobs or cleats and rubber hooks will not be approved.

(*f*) Must be so spliced or joined as to be both mechanically and electrically secure without solder. The joints must then be soldered, to insure preservation, and covered with an insulation equal to that on the conductors.

11. **Service Blocks:**

(*a*) Must be covered over their entire surface with at least two coats of waterproof paint.

(*b*) Telegraph, telephone, and similar wires must not be placed on the same cross-arm with electric light or power wires.

INTERIOR CONDUCTORS.

12. **All Interior Conductors:**

(*a*) Must be covered where they enter buildings from outside

terminal insulators to and through the walls, with extra waterproof insulation, and must have drip loops outside. The hole through which the conductor passes must be bushed with waterproof and non-combustible insulating tube or hard rubber tube, slanting upward toward the inside. The tube must be sealed with tape, thoroughly painted, and securing the tube to the wire.

(*b*) Must be arranged to enter and leave the building through a double contact service switch, which will effectually close the main circuit and disconnect the interior wires when it is turned "off." The switch must be so constructed that it shall be automatic in its action, not stopping between points when started, and prevent an arc between the points under all circumstances; it must indicate on inspection whether the current be "on" or "off," and be mounted in a non-combustible case, and kept free from moisture and easy of access to police or firemen.

(*c*) Must be always in plain sight, and never encased, except when *required* by the inspector.

(*d*) Must be covered in all cases with an *approved* non-combustible material that will adhere to the wire, not fray by friction, and bear a temperature of 150° F. without softening.

(*e*) Must be supported on glass or porcelain insulators, and kept rigidly at least eight inches from each other, except within the structure of lamps or on hanger boards, cut-out boxes, or the like, where less distance is necessary.

(*f*) Must be separated from contact with walls, floors, timbers, or partitions through which they may pass, by non-combustible insulating tube or hard rubber tube.

(*g*) Must be so spliced or joined as to be both mechanically and electrically secure without solder. They must then be soldered, to insure preservation, and covered with an insulation equal to that on the conductors.

LAMPS AND OTHER DEVICES.

13. **Arc Lamps.**—In every case:

(*a*) Must be carefully isolated from inflammable material.

(*b*) Must be provided at all times with a glass globe surrounding the arc, securely fastened upon a closed base. No broken or cracked globes to be used.

(*c*) Must be provided with an *approved* hand switch; also an automatic switch that will shunt the current around the carbons should they fail to feed properly.

(*d*) Must be provided with reliable stops to prevent carbons from falling out in case the clamps become loose.

(*e*) Must be carefully insulated from the circuit in all their exposed parts.

(*f*) Must be provided with a wire netting around the globe, and an *approved* spark arrester above to prevent escape of sparks, melted copper or carbon, where readily inflammable material is in the vicinity of the lamps. It is recommended that plain carbons, not copper plated, be used for lamps in such places.

(*g*) Hanger boards must be so constructed that all wires and current-carrying devices thereon shall be exposed to view, and thoroughly insulated by being mounted on a waterproof, non-combustible substance. All switches attached to the same must be so constructed that they shall be automatic in their action, not stopping between points when started, and preventing an arc between points under all circumstances.

14. **Incandescent Lamps in Series Circuits having a Maximum Potential of 300 Volts or Over:**

(*a*) Must be governed by the same rules as for arc lights, and each series lamp provided with an *approved* hand-spring switch and automatic cut-out.

(*b*) Must have each lamp suspended from a hanger board by means of a rigid tube.

(*c*) No electro-magnetic device for switches and no system of multiple-series or series multiple lighting will be approved.

(*d*) Under no circumstances can series lamps be attached to gas fixtures.

CLASS C.—INCANDESCENT (LOW PRESSURE) SYSTEMS.

300 *Volts or Less.*

OUTSIDE CONDUCTORS.

15. **Outside Overhead Conductors:**

(*a*) Must be erected in accordance with the rules for arc (series) circuit conductors.

(*b*) Must be separated not less than 12 inches, and be provided with an *approved* fusible cut-out, that will cut off the entire current as near as possible to the entrance to the building and inside the walls.

16. **Underground Conductors:**

(*a*) Must be protected against moisture and mechanical injury, and be removed at least two feet from combustible material when brought into a building, but not connected with the interior conductors.

(*b*) Must have a switch and a cut out for each wire between the underground conductors and the interior wiring when the two parts of the wiring are connected.

These switches and fuses must be placed as near as possible to the end of the underground conduit, and connected therewith by specially insulated conductors, kept apart not less than two and a half inches.

(*c*) Must not be so arranged as to shunt the current through a building around any catch-box.

INSIDE WIRING.

GENERAL RULES.

17. At the entrance of every building there shall be an *approved* switch placed in the service conductors by which the current may be entirely cut off.

18. **Conductors:**

(*a*) Must have an *approved* insulated covering, and must not be of sizes smaller than No. 14 B. & S , No. 16 B. W. G., or No. 4 E. S. G , except that in conduit installed under Rule 22, No. 16 B. & S., No. 18 B. W. G., or No. 4 E. S. G. may be used.

(*b*) Must be protected when passing through FLOORS ; or through walls, partitions, timbers. etc., in places liable to be exposed to dampness, by waterproof, non-combustible, insulating tubes, such as glass or porcelain.

Must be protected when passing through walls, partitions, timbers, etc., in places not liable to be exposed to dampness, by *approved* insulating bushings specially made for the purpose.

(*c*) Must be kept free from contact with gas, water, or other metallic piping, or any other conductors or conducting material which they may cross (except high potential conductors), by some continuous and firmly fixed non-conductor creating a separation of at least one inch. Deviations from this rule may sometimes be allowed by special permission.

(*d*) Must be so placed in crossing high potential conductors that there shall be a space of at least one foot at all points between the high and low tension conductors.

(*e*) Must be so placed in wet places that an air space will be left between conductors and pipes in crossing, and the former must be run in such a way that they cannot come in contact with the pipe accidentally. Wires should be run *over* all pipes upon which con-

densed moisture is likely to gather, or which by leaking might cause trouble on a circuit.

SPECIAL RULES.

19. Wiring not Encased in Moulding or Approved Conduit:

(*a*) Must be supported wholly on non-combustible insulators, constructed so as to prevent the insulating coverings of the wire from coming in contact with other substances than the insulating supports.

(*b*) Must be so arranged that wires of opposite polarity, with a difference of potential of 150 volts or less, will be kept apart at least two and one-half inches.

(*c*) Must have the above distance increased proportionately where a higher voltage is used, unless they are encased in moulding or *approved* conduit.

(*d*) Must not be laid in plaster, cement, or similar finish.

(*e*) Must never be fastened with staples.

In Unfinished Lofts, between Floor and Ceilings, in Partitions, and Other Places.

(*f*) Must have at least one inch clear air space surrounding them.

(*g*) Must be at least ten inches apart when possible, and should be run singly on separate timbers or studding.

(*h*) Wires run as above immediately under roofs, in proximity to water tanks or pipes, will be considered as exposed to moisture.

(*i*) Wires must not be fished for any great distance, and only in places where the inspector can satisfy himself that the above rules have been complied with.

(*j*) Twin wires must never be employed in this class of concealed work.

20. Mouldings:

(*a*) Must never be used in concealed work or in damp places.

(*b*) Must have at least two coats of waterproof paint or be impregnated with a moisture repellent.

(*c*) Must be made of two pieces, a backing and capping, so constructed as to thoroughly encase the wire, and maintain a distance of one half inch between conductors of opposite polarity, and afford suitable protection from abrasion.

21. Special Wiring:

In breweries, packing houses, stables, dyehouses, paper and pulp mills, or other buildings specially liable to moisture, or acid or

other fumes liable to injure the wires or insulation, except where used for pendants, conductors—

(*a*) Must be separated at least six inches.

(*b*) Must be provided with an *approved* waterproof covering.

(*c*) Must be carefully put up.

(*d*) Must be supported by glass or porcelain insulators. No switches or fusible cut-outs will be allowed where exposed to inflammable gases or dust, or to flyings of combustible material.

(*e*) Must be protected when passing through floors, walls, partitions, timbers, etc., by waterproof, non-combustible, insulating tubes, such as glass or porcelain.

22. **Interior Conduits:** *

(*a*) Must be continuous from one junction box to another, or to fixtures, and must be of material that will resist the fusion of the wire or wires they contain, without igniting the conduit.

(*b*) Must not be of such material or construction that the insulation of the conductor will ultimately be injured or destroyed by the elements of the composition.

(*c*) Must be first installed as a complete conduit system, without conductors, strings, or anything for the purpose of drawing in the conductors, and the conductors then to be pushed or fished in. The conductors must not be placed in position until all mechanical work on the building has been, as far as possible, completed.

(*d*) Must not be so placed as to be subject to mechanical injury by saws, chisels, or nails.

(*e*) Must not be supplied with a twin conductor or two separate conductors, in a single tube, unless the said two separate conductors or twin conductor, having an approved insulation, are enclosed in a complete, fully insulated, continuous iron conduit, and are in circuits installed as per table of Capacity of Wires (see Section 25), for currents not to exceed 100 amperes.

(*f*) *Must have all ends closed* with good adhesive material, either at junction boxes or elsewhere, whether such ends are concealed or exposed. Joints must be made air-tight and moisture-proof.

(*g*) Conduits must extend at least one inch beyond the finished surface of walls or ceilings until the mortar or other similar material be entirely dry, when the projection may be reduced to half an inch.

* The object of a tube or conduit is to facilitate the insertion or extraction of the conductors, to protect them from mechanical injury, and, as far as possible, from moisture. Tubes or conduits are to be considered merely as raceways, and are not to be relied on for insulation between wire and wire or between the wire and the ground.

23. Double Pole Safety Cut-outs:

(*a*) Must be in plain sight or enclosed in an *approved* box, readily accessible.

(*b*) Must be placed at every point where a change is made in the size of the wire (unless the cut-out in the larger wire will protect the smaller).

(*c*) Must be supported on bases of non-combustible, insulating, moisture-proof material.

(*d*) Must be supplied with a plug (or other device for enclosing the fusible strip or wire) made of non-combustible and moisture-proof material, and so constructed that an arc cannot be maintained across its terminals by the fusing of the metal.

(*e*) Must be so placed that on any combination fixture no group of lamps requiring a current of six amperes or more shall be ultimately dependent upon one cut-out. Special permission may be given *in writing* by the inspector for departure from this rule in case of large chandeliers.

(*f*) All cut-out blocks must be stamped with their *maximum* safe-carrying capacity in amperes, and *when installed* must be marked with the current they are intended to carry.

24. Safety Fuses:

(*a*) Must all be stamped or otherwise marked with the number of amperes they will carry indefinitely without melting.

(*b*) Must have fusible wires or strips (where the plug or equivalent device is not used), with contact surfaces or tips of harder metal, soldered or otherwise, having perfect electrical connection with the fusible part of the strip.

(*c*) Must all be so proportioned to the conductors they are intended to protect, that they will melt before the maximum safe-carrying capacity of the wire is exceeded.

25. Table of Capacity of Wires:

It must be clearly understood that the size of the fuse depends upon the size of the smallest conductor it protects, and not upon the amount of current to be used on the circuit. Below is a table showing the safe carrying capacity of conductors of different sizes in Birmingham, Brown & Sharp, and Edison gauges, which must be followed in the placing of interior conductors:

Brown & Sharp. Gauge No.	Amperes.	Birmingham. Gauge No.	Amperes.	Edison Standard. Gauge No.	Amperes.
0000	175	0000	175	200	175
000	145	000	150	180	160
00	120	00	130	140	135
0	100	0	110	110	110
1	95	1	95	90	95
2	70	2	85	80	85
3	60	3	75	65	75
4	50	4	65	55	65
5	45	5	60	50	60
6	35	6	50	40	50
7	30	7	45	30	40
8	25	8	35	25	35
10	20	10	30	20	30
12	15	12	20	12	20
14	10	14	15	8	15
16	5	16	10	5	10
18	3	18	5	3	5
		20	3	2	3

26. Switches:

(*a*) Must be mounted on moisture-proof and non-combustible bases, such as slate or porcelain.

(*b*) Must be double pole when the circuits which they control supply more than six 16-candlepower lamps, or their equivalent.

(*c*) Must have a firm and secure contact; must make and break readily and not stop when motion has once been imparted by the handle.

(*d*) Must have carrying capacity sufficient to prevent heating.

(*e*) Must be placed in dry, accessible places and be grouped as far as possible, being mounted—when practicable—upon slate or equally non combustible back boards. Jack-knife switches, whether provided with friction or spring stops, must be so placed that gravity will tend to open rather than close the switch.

FIXTURE WORK.

27. (*a*) In all cases where conductors are concealed within or attached to gas fixtures, the latter must be insulated from the gas-pipe system of the building by means of *approved* joints. The insulating material used in such joints must be of a substance not affected by gas, and that will not shrink or crack by variation in temperature. Insulating joints with soft rubber in their construction will not be approved.

(*b*) Supply conductors, and especially the splices to fixture wires,

must be kept clear of the grounded part of gas pipes, and where shells are used the latter must be constructed in a manner affording sufficient area to allow this requirement.

(*c*) When fixtures are wired outside, the conductors must be so secured as not to be cut or abraded by the pressure of the fastenings or motion of the fixture.

(*d*) All conductors for fixture work must have a waterproof insulation that is durable and not easily abraded, and must not in any case be smaller than No. 18 B. & S., No. 20 B. W. G., No. 2 E. S. G.

(*e*) All burrs or fins must be removed before the conductors are drawn into a fixture.

(*f*) The tendency to condensation within the pipes should be guarded against by sealing the upper end of the fixture.

(*g*) No combination fixture in which the conductors are concealed in a space less than one-fourth inch between the inside pipe and the outside casing will be approved.

(*h*) Each fixture must be tested for "contacts" between conductors and fixtures, for "short circuits," and for ground connections before the fixture is connected to its supply conductors.

(*i*) Ceiling blocks or fixtures should be made of insulating material; if not, the wires in passing through the plate must be surrounded with hard rubber tubing.

28. Arc Lights on Low Potential Circuits:

(*a*) Must be supplied by branch conductors not smaller than No. 12 B. & S. gauge.

(*b*) Must be connected with main conductors only through double pole cut outs.

(*c*) Must only be furnished with such resistances or regulators as are enclosed in non-combustible material, such resistances being treated as stoves.

Incandescent lamps must not be used for resistance devices.

(*d*) Must be supplied with globes and protected as in the case of arc lights on high potential circuits.

29. Electric Gas Lighting:

Where electric gas lighting is to be used on the same fixture with the electric light:

(*a*) No part of the gas piping or fixture shall be in electrical connection with the gas lighting circuit.

(*b*) The wires used with the fixtures must have a non-inflammable insulation, or, where concealed between the pipe and shell of the fixture, the insulation must be such as required for fixture wiring for the electric light.

(*c*) The whole installation must test free from "grounds."

(*d*) The two installations must test perfectly free from connection with each other.

30. **Sockets:**

(*a*) No portion of the lamp socket exposed to contact with outside objects must be allowed to come into electrical contact with either of the conductors.

(*b*) In rooms where inflammable gases may exist, or where the atmosphere is damp, the incandescent lamp and socket should be enclosed in a vapor-tight globe.

31. **Flexible Cord:**

(*a*) Must be made of conductors, each surrounded with a moisture-proof and a non-inflammable layer, and further insulated from each other by a mechanical separator of carbonizable material. Each of these conductors must be composed of several strands.

(*b*) Must not sustain more than one light, not exceeding 50-candlepower.

(*c*) Must not be used except for pendants, wiring of fixtures, and portable lamps or motors.

(*d*) Must not be used in show windows.

(*e*) Must be protected by insulating bushings where the cord enters the socket. The ends of the cord must be taped, to prevent fraying of the covering.

(*f*) Must be so suspended that the entire weight of the socket and lamp will be borne by knots under the bushing in the socket, and above the point where the cord comes through the ceiling block or rosette, in order that the strain may be taken from the joints and binding screws.

(*g*) Must be equipped with keyless sockets as far as practicable, and be controlled by wall switches.

[Classes D and E, relating to Alternating System and Electric Railways, are here omitted.]

MISCELLANEOUS.

44. *a.* The wiring in any building must test free from grounds; *i.e.*, each main supply line and every branch circuit shall have an insulation resistance of at least 25,000 ohms, and should have an insulation resistance between conductors and between all conductors and the ground (not including attachments, sockets, receptacles, etc.) of not less than the following:

Up to	10	amperes	4,000,000
"	25	"	1,600,000
"	50	"	800,000
"	100	"	300,000
"	200	"	160,000
"	400	"	80,000
"	800	"	22,000
"	1,600	"	11,000

All cut-outs and safety devices in place in the above.

Where lamp sockets, receptacles, and electroliers, etc., are connected, one-half of the above will be required.

(*b*) Ground wires for lightning arresters of all classes, and ground detectors, must not be attached to gas pipes within the building.

(*c*) Where telephone, telegraph, or other wires connected with outside circuits are bunched together within any building, or where inside wires are laid in conduit or duct with electric light or power wires, the covering of such wires must be fire resisting, or else the wires must be enclosed in an air-tight tube or duct.

(*d*) All conductors connecting with telephone, district messenger, burglar alarm, watch clock, electric time, and other similar instruments, must be provided near the point of entrance to the building with some protective device which will operate to shunt the instruments in case of a dangerous rise of potential, and will open the circuit and arrest an abnormal current flow. Any conductor normally forming an innocuous circuit may become a source of fire hazard if crossed with another conductor, through which it may become charged with a relatively high pressure.

(*e*) The following formula for soldering fluid is suggested:

Saturated solution of zinc	5 parts.
Alcohol	4 parts.
Glycerine	1 part.

ADDENDA.

Underground Conductors (see Rule 16):

Must end outside of the main walls of the building, and not be brought into a building where it is possible to avoid it; and when brought into the building, or any vault or area connected with same, must be removed at least *two* feet from all combustible material, and kept free and clear of contact with any conducting material.

Testing:

The Rules and all existing regulations of the local authorities in reference to the stringing of wires must be strictly observed.

DEFINITIONS.

DEFINITIONS OF THE WORD "APPROVED" AS USED IN THE RULES FOR ELECTRIC WIRING.

Rule 10. Outside Conductors:

Section *a*. Insulation that will be *approved* for service wires must be solid, at least $\frac{3}{64}$ of an inch in thickness, and covered with a substantial braid. It must not readily carry fire, must show an insulating resistance of one megohm per mile after two weeks' submersion in water at 70° F., and three days' submersion in lime water, with a current of 550 volts and after three minutes' electrification.

Rule 12. Interior Conductors:

Section *d*. Insulation that will be *approved* for interior conductors must be solid, at least $\frac{3}{64}$ of an inch in thickness, and covered with a substantial braid. It must not readily carry fire, must show an insulating resistance of one megohm per mile after two weeks' submersion in water at 70° F., and three days' submersion in lime water, with a current of 550 volts and after three minutes' electrification.

Rule 13. Arc Lamps:

Section *c*. The hand switch to be *approved*, if placed anywhere except on the lamp itself, must comply with the requirements for switches on hanger boards as laid down in new Section *g* of Rule 13.

Rule 13. Arc Lamps:

Section *f*. An *approved* spark arrester is one which will so close the upper orifice of the globe that it will be impossible for any sparks thrown off by the carbons to escape.

Rule 15. Outside Overhead Conductors:

Section *b*. An *approved* fusible cut-out must comply with the sections of Rules 23 and 24 describing fuses and cut-outs.

Rule 17:

The switch required by this rule to be *approved* must be double pole, must plainly indicate whether the current is "on" or "off,"

and must comply with Sections *a, c, d,* and *e* of Rule 26 relating to switches.

Rule 18. Conductors:

Section *a*. In so-called "concealed" wiring, moulding, and conduit work, *and* in places liable to be exposed to dampness, the insulating covering of the wire, to be *approved*, must be solid, at least $\frac{3}{64}$ of an inch in thickness, and covered with a substantial braid. It must not readily carry fire, must show an insulating resistance of one megohm per mile after two weeks' submersion in water at 70° F., and three days' submersion in lime water, with a current of 550 volts and after three minutes' electrification.

For work which is *entirely* exposed to view throughout the whole interior circuits, and not liable to be exposed to dampness, a wire with an insulating covering that will not support combustion, will resist abrasion, is at least $\frac{1}{16}$ of an inch in thickness, and thoroughly impregnated with a moisture repellent, will be *approved.*

Rule 18. Conductors:

Section *b*, second paragraph. Except for FLOORS, *and* for places liable to be exposed to dampness, Glass, Porcelain, *metal-sheathed* Interior Conduit, and Vulca Tube, when made especially for bushings, will be *approved. The two last named will not be approved if cut from the usual lengths of tube made for conduit work, nor when made without a head or flange on one end.*

Rule 21. Special Wiring:

Section *b*. The insulating covering of the wire to be *approved* under this section must be solid, at least $\frac{3}{64}$ of an inch in thickness, and covered with a substantial braid. It must not readily carry fire, must show an insulating resistance of one megohm per mile after two weeks' submersion in water at 70° F., and three days' submersion in lime water, with a current of 550 volts after three minutes' electrification, and must *also* withstand a satisfactory test against such chemical compounds or mixtures as it will be liable to be subjected to in the risk under consideration.

Rule 23. Double Pole Safety Cut-outs:

Section *a*. To be *approved*, boxes must be constructed, and cut-outs arranged, whether in a box or not, so as to obviate any danger of the melted fuse metal coming in contact with any substance which might be ignited thereby.

Rule 27. Fixture Work:

Section *a*. Insulating joints to be *approved* must be entirely made of material that will resist the action of illuminating gases, and will not give way or soften under the heat of an ordinary gas

flame. They shall be so arranged that a deposit of moisture will not destroy the insulating effect, and shall have an insulating resistance of 250,000 ohms between the gas-pipe attachments, and be sufficiently strong to resist the strain they will be liable to in attachment.

Notice of the Approval of Certain Wires and Materials, and the Interpretation of Certain Rules.

Rule 4. Switch-boards:

Section *a*. Special attention is called to the fact that switch-boards should not be built down to the floor, nor up to the ceiling, but a space of at least eighteen inches, or two feet, should be left between the floor and the board, and between the ceiling and the board, in order to prevent fire from communicating from the switch-board to the floor or ceiling, and also to prevent the forming of a partially concealed space very liable to be used for storage of rubbish and oily waste.

Rule 5. Resistance Boxes:

Section *a*. The word "frame" in this section relates to the entire case and surrounding of the rheostat, and not alone to the upholding supports.

Rule 9. Resistance Boxes:

Section *a*. The word "frame" in this section relates to the entire case and surrounding of the rheostat, and not alone to the upholding supports.

Class B:

Any circuit attached to any machine, or combination of machines, which develop over 300 volts difference of potential between any two wires, shall be considered as a high potential circuit and coming under that class, unless an *approved* transforming device is used, which cuts the difference of potential down to less than 300 volts.

Rule 10. Outside Conductors:

Section *f*. All joints must be soldered, even if made with the McIntyre or any other patent splicing device. This ruling applies to joints and splices in all classes of wiring covered by these Rules.

Rule 15. Outside Overhead Conductors:

Section *b*. The cut-out required by this section must be placed so as to protect the switch, required by Rule 17.

Rule 16. Underground Conductors:

Section *b*. The cut-out required by this section must be placed so as to protect the switch.

Rule 22. Interior Conduits:

The American Circular Loom Co.'s Tube, the *metal-sheathed* Interior Conduit Tube, and the Vulca Tube are approved for the class of work called for in this rule.

FRENCH PLATE WINDOW-GLASS.

lished French plate window-glass, which is cond to be the highest grade of window-glass in the market, may ained in lights varying in size from a piece one inch square ight eight feet wide and fourteen feet long. Owing to the cost of rolling large lights, the price per square foot of large is sometimes twice that of smaller lights ; so that the cost of glass must be estimated by a price-list, giving the cost of every nt size of light. Such a price-list is given below. This list ıs the same from year to year, and is known as the "standist for polished plate glass. The fluctuations in the price of ıre arranged by means of a discount, which is the same for -s. At the present time the discount on large lots of plates about fifty per cent.

weight of polished plate-glass averages $3\frac{1}{2}$ pounds per square

APPROXIMATE WEIGHT OF POLISHED PLATE GLASS BOXED.

end the glass $3\frac{1}{2}$ pounds per square foot. Weight of box the contents of a plate of greatest width and length of those l therein, multiplied by 10. Thus:

$$\left.\begin{array}{l}\text{plate } 36'' \times 96'' \\ \text{plate } 60'' \times 84''\end{array}\right\} = 59 \text{ feet} \times 3\tfrac{1}{2} = 206\tfrac{1}{2} \text{ pounds.}$$

$$\text{ze of box } 60'' \times 96'' = 40 \text{ feet} \times 10 = 400 \text{ pounds.}$$

$$606\tfrac{1}{2} \text{ pounds.}$$

Price-List of Polished Plate-Glass.

Sizes, in inches; prices, in dollars and cents.

	12	14	16	18	20	22	24	26	28
24	2.00	2.35	2.70	3.05	4.05	4.45	4.90	–	–
26	2.20	2.55	2.90	3.95	4.40	4.85	5.30	5.70	–
28	2.35	2.75	3.80	4.25	4.75	5.20	5.70	8.15	8.75
30	2.55	2.95	4.05	4.55	5.10	5.60	6.10	8.70	9.40
32	2.70	3.80	4.35	4.90	5.40	5.95	8.60	9.30	10.00
34	2.85	4.05	4.60	5.20	5.75	8.35	9.10	9.90	10.65
36	3.05	4.25	4.90	5.50	6.10	8.85	9.65	10.45	11.25
38	3.85	4.50	5.15	5.80	8.50	9.35	10.20	11.05	11.90
40	4.05	4.75	5.40	6.10	8.95	9.85	10.75	11.65	12.50
42	4.25	5.00	5.70	8.45	9.40	10.35	11.25	12.20	13.15
44	4.45	5.20	5.95	8.85	9.85	10.80	11.80	12.80	13.75
46	4.70	5.45	8.20	9.20	10.30	11.30	12.35	13.35	14.40
48	4.90	5.70	8.60	9.65	10.75	11.80	12.90	13.95	15.05
50	5.10	5.95	8.95	10.05	11.15	12.30	13.40	14.55	15.65
52	5.30	8.15	9.30	10.45	11.65	12.80	13.95	15.10	21.55
54	5.50	8.45	9.65	10.85	12.05	13.30	14.50	15.70	22.35
56	5.70	8.75	10.00	11.25	12.50	13.75	15.05	21.55	23.20
58	5.90	9.10	10.35	11.65	12.95	14.25	15.55	22.30	24.00
60	6.10	9.40	10.75	12.05	13.40	14.75	16.10	23.10	24.85
62	8.30	9.70	11.10	12.45	13.85	15.25	22.00	23.85	25.70
64	8.55	10.00	11.45	12.90	14.30	15.75	22.70	24.60	26.50
66	8.85	10.35	11.80	13.30	14.75	21.50	23.45	25.40	27.35
68	9.10	10.65	12.15	13.70	15.20	22.15	24.15	26.15	28.15
70	9.40	10.95	12.55	14.10	15.65	22.80	24.85	26.90	29.00
72	9.65	11.25	12.90	14.50	16.10	23.45	25.55	27.70	29.80
74	9.95	11.60	13.25	14.90	21.90	24.10	26.25	28.45	30.65
76	10.20	11.90	13.60	15.30	22.50	24.75	27.00	29.25	31.50
78	10.45	12.20	14.05	15.70	23.10	25.40	27.70	30.00	32.30
80	10.75	12.50	14.30	16.10	23.65	26.05	28.40	30.75	33.15
82	11.00	12.85	14.65	21.85	24.25	26.70	28.60	31.55	33.95
84	11.25	13.15	15.05	22.35	24.85	27.35	29.80	32.30	34.80
86	[illegible]	13.45	15.40	22.90	25.45	28.00	30.55	33.10	35.60
88	11.80	13.75	15.75	23.45	26.05	28.65	31.25	33.85	36.45
90	12.05	14.10	16.10	23.95	26.65	29.30	31.95	34.60	37.30
92	[illegible]	14.40	21.80	24.50	27.20	29.95	32.65	35.40	38.10
94	[illegible]	14.70	22.25	25.05	27.80	30.60	33.35	36.15	38.95
96	12.90	15.05	22.70	25.55	28.40	31.25	34.10	36.90	39.75
98	13.15	15.35	23.20	26.10	29.00	31.90	34.80	37.70	40.60
100	13.40	15.65	23.65	26.65	29.60	32.55	35.50	38.45	41.40
102	[illegible]	15.95	24.15	27.15	30.20	33.20	36.20	39.25	42.20
104	[illegible]	[illegible]	24.60	27.70	30.75	33.85	36.90	40.00	43.10
106	[illegible]	[illegible]	[illegible]	28.20	31.35	[illegible]	37.65	40.75	43.90
108	[illegible]	22.35	[illegible]	28.75	31.95	[illegible]	38.35	41.55	[illegible]
110	[illegible]	22.80	26.05	29.30	[illegible]	35.80	39.05	42.30	[illegible]
112	[illegible]	[illegible]	[illegible]	29.80	[illegible]	36.45	39.75	43.10	[illegible]
114	[illegible]	[illegible]	27.00	30.35	[illegible]	[illegible]	40.45	43.85	47.20
116	[illegible]	24.00	27.45	30.90	[illegible]	[illegible]	41.20	44.60	48.05
118	[illegible]	[illegible]	[illegible]	31.50	[illegible]	[illegible]	41.90	45.40	[illegible]
120	[illegible]	[illegible]	28.40	[illegible]	[illegible]	[illegible]	42.60	46.15	49.70
122	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	46.90	[illegible]
124	[illegible]	[illegible]	[illegible]	[illegible]	36.70	[illegible]	44.00	47.70	[illegible]
126	[illegible]	26.10	29.80	[illegible]	37.30	41.00	44.75	48.45	52.20
128	[illegible]	[illegible]	30.30	31.10	[illegible]	[illegible]	45.45	49.25	53.00
	12	14	16	18	20	22	24	26	28

Price-List of Polished Plate-Glass (*Continued*).

In dollars and cents.

42	44	46	48
–	–	–	–
–	–	–	–
	–	–	–
–	–	–	–
26.10	–	–	–
27.35	28.65	–	–
28.60	29.95	31.30	–
29.80	31.25	32.65	34.10
31.05	32.55	34.00	35.50
32.30	33.85	35.40	36.90
33.55	35.15	36.75	38.35
34.80	36.45	38.10	39.75
36.05	37.75	39.45	41.20
37.30	39.05	40.85	42.60
38.50	40.35	42.20	44.00
39.75	41.65	43.55	45.45
41.00	42.95	44.90	46.85
42.25	44.25	46.25	48.30
43.50	45.55	47.65	49.70
44.75	46.85	49.00	51.10
45.95	48.15	50.35	52.55
47.20	49.45	51.70	53.95
48.45	50.75	53.05	55.40
49.70	52.05	54.45	56.80
50.95	53.35	55.80	58.20
52.20	54.65	57.15	59.65
53.45	55.95	58.50	61.05
54.65	57.30	59.90	62.50
55.90	58.60	61.25	63.90
57.15	59.90	62.60	65.30
58.40	61.20	63.95	66.75
59.65	62.50	65.30	68.15
60.90	63.80	66.70	69.60
62.15	65.10	68.05	71.00
63.35	66.40	69.40	72.40
64.60	67.70	70.75	73.85
65.85	69.00	72.15	75.25
67.10	70.30	73.50	76.70
68.35	71.60	74.85	78.10
69.60	72.90	76.20	79.50
70.80	74.20	77.55	80.95
72.05	75.50	78.95	82.35
73.30	76.80	80.30	83.80
74.55	78.10	81.65	85.20
75.80	79.40	83.00	91.00
77.05	80.70	84.35	93.00
78.30	82.00	90.00	94.00
79.50	83.30	92.00	96.00
80.75	84.60	93.00	97.00
82.00	90.00	94.00	99.00
83.25	92.00	96.00	100.00
84.50	93.00	97.00	102.00
90.00	94.00	99.00	103.00
91.00	96.00	100.00	105.00
93.00	97.00	102.00	106.00
94.00	99.00	103.00	108.00
42	44	46	48

PRICE-LIST OF POLISHED PLATE-GLASS (*Continued*).

Sizes, in inches; prices, in dollars and cents.

	50	52	54	56	58	60	62	64	66	68	70	72
50	37	–	–	–	–	–	–	–	–	–	–	–
52	38	40	–	–	–	–	–	–	–	–	–	–
54	40	42	43	–	–	–	–	–	–	–	–	–
56	41	43	45	46	–	–	–	–	–	–	–	–
58	43	45	46	48	50	–	–	–	–	–	–	–
60	44	46	48	50	51	53	–	–	–	–	–	–
62	46	48	50	51	53	55	57	–	–	–	–	–
64	47	49	51	53	55	57	59	61	–	–	–	–
66	49	51	53	55	57	59	61	62	64	–	–	–
68	50	52	54	55	58	60	62	64	66	68	–	–
70	52	54	56	58	60	62	64	66	68	70	72	–
72	53	55	58	60	62	64	66	68	70	72	75	77
74	55	57	59	61	63	66	68	70	72	74	77	79
76	55	58	61	63	65	67	70	72	74	76	79	81
78	58	60	62	65	67	69	72	74	76	78	81	83
80	59	62	64	66	69	71	73	76	78	80	83	85
82	61	63	65	68	70	73	75	78	80	82	85	92
84	62	65	67	70	72	75	77	80	82	84	91	94
86	64	66	69	71	74	76	79	81	84	91	94	96
88	65	68	70	73	76	78	81	83	90	93	96	99
90	67	69	72	75	77	80	83	85	92	95	98	101
92	68	71	73	76	79	82	84	92	94	97	100	103
94	70	72	75	78	81	83	91	94	97	99	102	105
96	71	74	77	80	82	85	93	96	99	102	105	108
98	72	75	78	81	84	91	95	98	101	104	107	110
100	74	77	80	83	90	93	96	100	103	106	109	112
102	75	78	81	84	92	95	98	102	105	108	111	114
104	77	80	83	91	94	97	100	104	107	110	113	116
106	78	82	85	92	96	99	102	106	109	112	115	119
108	80	83	91	94	97	101	104	108	111	114	118	121
110	81	85	92	96	99	103	106	110	113	116	120	123
112	[illegible]	91	94	98	101	105	108	112	115	118	122	125
114	[illegible]	92	96	[illegible]	103	105	110	113	117	121	124	128
116	[illegible]	94	97	101	105	108	112	115	119	123	126	130
118	[illegible]	[illegible]	[illegible]	103	105	110	114	117	121	125	128	132
120	[illegible]	97	101	[illegible]	108	112	116	119	123	127	131	134
122	[illegible]	[illegible]	102	[illegible]	110	114	118	121	125	129	133	137
124	[illegible]	100	104	108	112	115	120	123	127	131	135	139
126	[illegible]	102	[illegible]	110	111	118	122	125	129	133	137	141
128	[illegible]	104	[illegible]	112	115	119	123	127	131	135	139	143
130	101	[illegible]	[illegible]	113	117	121	125	129	133	138	142	146
132	[illegible]	107	111	115	119	123	127	131	136	140	144	148
134	[illegible]	108	113	117	121	[illegible]	129	133	138	142	146	150
136	[illegible]	110	114	118	123	127	131	135	140	144	148	152
138	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	133	137	142	146	[illegible]	[illegible]
140	[illegible]	[illegible]	118	[illegible]	125	[illegible]	[illegible]	139	144	148	152	157
142	[illegible]	[illegible]	[illegible]	[illegible]	128	[illegible]	137	141	146	150	[illegible]	[illegible]
144	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	139	143	148	152	157	161
146	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	141	[illegible]	150	154	[illegible]	[illegible]
148	[illegible]	[illegible]	123	[illegible]	134	[illegible]	[illegible]	147	[illegible]	157	161	[illegible]
150	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	154	159	[illegible]	[illegible]
152			[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	174	[illegible]	[illegible]
154	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	171	176	181	[illegible]
156	[illegible]	[illegible]	[illegible]	147	[illegible]	[illegible]	[illegible]	[illegible]	173	178	184	[illegible]
158		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	170	175	181	[illegible]	191
160	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	161	167	172	177	183	188	194
162	[illegible]	[illegible]	147	[illegible]	[illegible]	[illegible]	169	174	180	185	191	196
164		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	171	176	182	187	193	198
	50	52	54	56	58	60	62	64	66	68	70	72

Price-List of Polished Plate-Glass (*Concluded*).

Sizes, in inches; prices, in dollars.

The above table was kindly furnished the author by Messrs. Hills, Turner, & Boston, Mass., importers and dealers in French and American window-glass

Ordinary Window-Glass.

Window-glass is sold by the box, which contains, as nearly as may be, fifty square feet, whatever may be the size of the panes.

The thickness of ordinary, or "single thick," window-glass, is about one-sixteenth of an inch, and, of "double thick," nearly one-eighth of an inch.

The tensile strength of common glass varies from 2000 pounds to 3000 pounds per square inch, and its crushing strength from 6000 pounds to 10,000 pounds.

The following table gives the *number of panes of window-glass in one box, or fifty feet:*—

Size, in inches.	Panes in box.	Size, in inches.	Panes in box.	Size, in inches.	Panes in box.	Size, in inches.	Panes in box.
6 × 8	150	12 × 19	32	16 × 20	23	24 × 44	7
7 × 9	115	12 × 20	30	16 × 22	20	24 × 50	6
8 × 10	90	12 × 21	29	16 × 24	19	24 × 56	5
8 × 11	82	12 × 22	27	16 × 30	15	26 × 36	8
8 × 12	75	12 × 23	26	16 × 36	12	26 × 40	7
9 × 10	80	12 × 24	25	16 × 40	11	26 × 48	6
9 × 11	72	13 × 14	40	18 × 20	20	26 × 54	5
9 × 12	67	13 × 15	37	18 × 22	18	28 × 34	8
9 × 13	62	13 × 16	35	18 × 24	17	28 × 40	6
9 × 14	57	13 × 17	33	18 × 26	15	28 × 46	6
9 × 15	53	13 × 18	31	18 × 34	12	28 × 50	5
9 × 16	50	13 × 19	29	18 × 36	11	30 × 40	6
10 × 10	72	13 × 20	28	18 × 40	10	30 × 44	4
10 × 12	60	13 × 21	26	18 × 44	9	30 × 48	5
10 × 13	55	13 × 22	25	20 × 22	16	30 × 54	5
10 × 14	52	13 × 24	23	20 × 24	15	32 × 42	5
10 × 15	48	14 × 15	34	20 × 25	14	32 × 44	5
10 × 16	45	14 × 16	32	20 × 26	14	32 × 46	5
10 × 17	42	14 × 18	29	20 × 28	13	32 × 48	5
10 × 18	40	14 × 19	27	20 × 30	12	32 × 50	4
11 × 11	59	14 × 20	26	20 × 34	11	32 × 54	4
11 × 12	55	14 × 22	23	20 × 38	10	32 × 56	4
11 × 13	50	14 × 24	22	20 × 40	9	32 × 60	4
11 × 14	47	14 × 28	18	20 × 44	8	34 × 40	5
11 × 15	44	14 × 32	16	20 × 50	7	34 × 44	5
11 × 16	41	14 × 36	14	22 × 24	14	34 × 46	5
11 × 17	39	14 × 40	13	22 × 26	13	34 × 50	4
11 × 18	36	15 × 16	30	22 × 28	12	34 × 52	4
12 × 12	50	15 × 18	27	22 × 36	9	34 × 56	4
12 × 13	46	15 × 20	24	22 × 40	8	36 × 44	5
12 × 14	43	15 × 22	22	22 × 50	7	36 × 50	4
12 × 15	40	15 × 24	20	24 × 28	11	36 × 56	4
12 × 16	38	15 × 30	16	24 × 30	10	36 × 60	3
12 × 17	35	15 × 32	15	24 × 32	9	36 × 64	3
12 × 18	33	16 × 18	25	24 × 36	8	40 × 60	3

Glass for Skylights.

Where skylights are glazed with clear or double thick glass, it may be used in lengths of from sixteen to thirty inches by a width of from nine to fifteen inches. A lap of at least an inch and a half

cessary for all joints. This is the cheapest mode of glazing. best, however, for skylight purposes, is fluted or rough plate- . The following thicknesses are recommended as proportion- ɔ sizes: —

inches by	48	inches is the extent for glass			$\frac{3}{16}$	inch	thickness.
"	60	"	"	"	$\frac{1}{4}$	"	"
"	100	"	"	"	$\frac{3}{8}$	"	"
"	156	"	"	"	$\frac{1}{2}$	"	"

Weight of Rough Glass per Square Foot.

ıickness	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	1	inch.
ɜight	2	$2\frac{1}{2}$	$3\frac{1}{2}$	5	7	$8\frac{1}{2}$	10	$12\frac{1}{2}$	pounds.

ASPHALTUM.

phaltum is used extensively for composition roofing, for the purpose as tar.

phaltum, or solid bitumen, is a natural pitch, found in differ- ountries. The most accessible and economical for use in the ed States is obtained from the " Great Pitch Lake," a remark- and inexhaustible deposit in the island of Trinidad.

is impervious to water, and is one of the most unchangeable durable substances known, — qualities which, together with ınacity, adhesiveness, and resistance to the effects of the most ɜme changes of heat and cold, make it a cementing material of ɽreatest value for roofs, pavements, and various other purposes.

ıe principal advantages claimed for asphaltum as a roofing ɪrial over pitch and coal-tar, arise from the fact that the bitu- ɪus matter of the asphalt is not volatile at any temperature of un's heat, and is therefore permanent; while in all materials ıfactured from coal-tar there are volatile oils, which slowly orate on exposure to the sun and air, destroying the flexibility life of the material. The fact is now well known, that any ı or cement manufactured from coal-tar thus gradually deteri- ɜs, until, in the course of years, it becomes brittle, and crum- away; and that felt saturated with coal-tar in like manner ens, until it becomes brittle and finally worthless.

ɔphalted sheathing-felt, for roofing purposes, and for g under shingles, slates, clapboards, etc., is also made in a ar manner to the tarred papers more commonly used for the e purposes. Both these materials may be found in the mar- in a condition ready for use.

of view, asphalt is without a peer. Its surface is smooth, regular, and non-absorbent, with no cavities or cracks of any kind to retain the infected mud and dust of the streets, and the soil beneath it is kept dry. It is more thoroughly cleaned, either by sweeping or washing, than any other pavement. Its freedom from noise, and its other excellences, are fast placing it in all the business and banking streets of the city of London, where it seems to be superseding all other pavements. In comparison with granite, its great economy is to brain-workers and the owners of horses." In an article in Johnson's Cyclopædia, Gen. Q. A. Gillmore says of the "natural rock asphalt," —

"It must be conceded that nothing has yet been discovered which can replace with entire satisfaction the bituminous limestones of Seyssel and Val de Travers and Sicily. In the natural asphaltic rock, the calcareous matter is so intimately and impalpably combined with the bitumen, resists so thoroughly the action of air and water and even muriatic acid, is so entirely free from moisture,—properties due, perhaps, to the vast pressure and intense heat under which the ingredients have been incorporated by nature,—that we are forced to attribute the excellence of this material to the existence of certain natural conditions which the most skilful artificial methods fail to reproduce."

Mastic asphalt is used for floors of cellars, stores, breweries, malt-houses, hotel kitchens, stables, laundries, conservatories, public buildings, carriage-factories, sugar-refineries, mills, rinks, etc.; and for any place where a hard, smooth, clean, dry, fire and water proof, odorless, and durable covering of a light color is required, either in basement or upper stories. It can be laid either over cement concrete, brick, or wood, in one sheet without seams: also over cement concrete for roofs, for fire-proof buildings. For dwelling-house cellars, especially on moist or filled land, this material is especially adapted, being water-tight, non-absorbent, free from mould or dust, impervious to sewer-gases, and for sanitary purposes invaluable.

Mastic asphalt is also valuable for *damp courses* over foundations, and for covering vaults and arches underground.

The use of asphalt for *roofs* is extending, many of the principal buildings in London and a large number in this country being covered with it. It possesses especial advantages for this purpose from the fact that it is both fireproof and fire-resisting.

Architects and builders desiring to employ asphalt for any of the above purposes should be careful to secure the genuine *Val-de-Travers* or *Seyssel* or *Sicilian* rock asphalt, as there are imitations which are of but little value.

For floors of cellars, courtyards, etc., laid on the ground, a base of cement concrete 3 inches thick should first be laid; and over this is put a layer of asphalt from $\frac{3}{4}$ to $1\frac{1}{2}$ inch thick, according to the use to which it is to be put. For ordinary cellar floors, the asphalt need not be more than $\frac{3}{4}$ inch thick: for yards on which heavy teams are to drive, it should be $1\frac{1}{2}$ inches thick. In specifying asphalt pavement, both the thickness of the concrete and of the asphalt should be given: it should also be remembered, "asphalt pavement" does not include the concrete foundation unless so specified.

In laying asphalt over plank or boards, a layer of stout, *dry* (not tarred) sheathing-paper should first be put down, and the asphalt laid on this. Asphalt floors for stables should be at least 1 inch thick. The *cost* of rock asphalt in the large cities varies from 15 to 20 cents per square foot in jobs of 2,000 feet and over. This does not include the concrete foundation. Imitation asphalts are laid for considerably less, and German and other cheap asphalts for about two-thirds the above price.

CAPACITY OF FREIGHT CARS.

[From the "American Architect."]

A car-load is nominally 20,000 pounds. It is also 70 barrels of salt, 70 of lime, 90 of flour, 60 of whiskey, 200 sacks of flour, 6 cords of soft wood, 18 to 20 head of cattle, 50 to 60 head of hogs, 80 to 100 head of sheep, 9000 feet of solid boards, 17,000 feet of siding, 13,000 feet of flooring, 40,000 shingles, one-half less of hard lumber, one-fourth less of green lumber, one-tenth of joists, scantling, and all other large timbers, 340 bushels of wheat, 400 of corn, 680 of oats, 400 of barley, 300 of flax-seed, 360 of apples, 430 of Irish potatoes, 360 of sweet potatoes, 1000 bushels of bran.

WEIGHT OF A CUBIC FOOT OF SUBSTANCES.

NAMES OF SUBSTANCES.	Average weight, in lbs.
Anthracite, solid, of Pennsylvania	93
" broken, loose	54
" " moderately shaken	58
" heaped bushel, loose	80
Ash, American white, dry	38
Asphaltum	87
Brass (copper and zinc), cast	504
" rolled	524
Brick, best pressed	150
" common hard	125
" soft, inferior	100
Brickwork, pressed brick	140
" ordinary	112
Cement, hydraulic, ground, loose, American, Rosendale	56
" hydraulic, ground, loose, American, Louisville	50
" hydraulic, ground, loose, English, Portland,	90
Cherry, dry	42
Chestnut, dry	41
Coal, bituminous, solid	84
" " broken, loose	49
" " heaped bushel, loose	74
Coke, loose, of good coal	27
" " heaped bushel	38

Weight of Cubic Foot of Substances (Continued).

Names of Substances.	Average weight, in lbs.
Copper, cast	542
" rolled	548
Earth, common loam, dry, loose	76
" " " " moderately rammed	95
" as a soft flowing mud	108
Ebony, dry	76
Elm, dry	35
Flint	162
Glass, common window	157
Gneiss, common	168
Gold, cast, pure or 24-carat	1204
" pure, hammered	1217
Granite	170
Gravel, about the same as sand	90 to 106
Hemlock, dry	25
Hickory, dry	53
Hornblende, black	203
Ice	58.7
Iron, cast	450
" wrought, purest	485
" " average	480
Ivory	114
Lead	711
Lignum vitæ, dry	83
Lime, quick, ground, loose, or in small lumps	53
" " " " thoroughly shaken	75
" " " " per struck bushel	66
Limestones and marbles	168
" " " loose, in irregular fragments,	96
Mahogany, Spanish, dry	53
" Honduras, dry	35
Maple, dry	49
Marbles. (See Limestones.)	
Masonry, of granite or limestone, well-dressed	165
" " mortar rubble	154
" " dry rubble	138
" " sandstone, well-dressed	144
Mercury, at 32° Fahrenheit	849

Weight of Cubic Foot of Substances (Concluded).

Names of Substances.	Average weight, in lbs.
Mica	182
Mortar, hardened	103
Mud, dry, close	80 to 110
" wet, fluid, maximum	120
Oak, live, dry	59
" white, dry	52
" other kinds	32 to 45
Petroleum	55
Pine, white, dry	25
" yellow, Northern	34
" " Southern	45
Platinum	1342
Quartz, common, pure	165
Rosin	69
Salt, coarse, Syracuse, N.Y.	45
" Liverpool, fine, for table use	49
Sand, of pure quartz, dry, loose	90 to 106
" well shaken	99 to 117
" perfectly wet	120 to 140
Sandstones, fit for building	151
Shales, red or black	162
Silver	655
Slate	175
Snow, freshly fallen	5 to 12
" moistened and compacted by rain	15 to 50
Spruce, dry	25
Steel	490
Sulphur	125
Sycamore, dry	37
Tar	62
Tin, cast	459
Turf or peat, dry, unpressed	20 to 30
Walnut, black, dry	38
Water, pure rain or distilled, at 60 degrees F.	62½
" sea	64
Wax, bees'	60.5
Zinc or spelter	437

Green timbers usually weigh from one-fifth to one-half more than dry.

DIMENSIONS AND WEIGHT OF CHURCH BELLS.

MANUFACTURED BY WILLIAM BLAKE & CO., BOSTON.

WEIGHT.	Tone.	Diameter.	Size of frame, outside. Horizontal dimensions.	Diameter of vertical wheel.
lbs.				
200		21 in.	42 × 32 in.	34 in.
250		22½ in.	46 × 36 in.	38 in.
300	E	24 in.	46 × 36 in.	38 in.
350	D♯	26 in.	46 × 36 in.	38 in.
400	D	27½ in.	53 × 40 in.	44 in.
500	C♯	29 in.	53 × 40 in.	44 in.
600	C	31 in.	60 × 48 in.	49 in.
700	B	33 in.	60 × 48 in.	49 in.
800	A♯	34¼ in.	60 × 48 in.	49 in.
900		36 in.	70 × 54 in.	58 in.
1000	A	37 in.	70 × 54 in.	58 in.
1100	G♯	38½ in.	76 × 57 in.	64 in.
1200		39 in.	76 × 57 in.	64 in.
1300		40 in.	76 × 57 in.	64 in.
1400	G	41 in.	76 × 57 in.	64 in.
1500		42 in.	76 × 57 in.	64 in.
1600		43½ in.	89 × 63 in.	72 in.
1700	F♯	44½ in.	89 × 63 in.	72 in.
1850	F	46 in.	89 × 63 in.	72 in.
2000		47 in.	91 × 67 in.	75 in.
2200	E	48 in.	91 × 67 in.	75 in.
2500	D♯	51 in.	100 × 70 in.	84 in.
3000		53 in.	112 × 73 in.	96 in.
3200	D	55 in.	112 × 73 in.	96 in.
4000	C♯	58 in.	124 × 78 in.	108 in.
5000	C	63 in.	124 × 78 in.	108 in.

SIZE OF ROPE FOR BELLS.

For bells of less than 500 pounds . . . . . ½ **inch diameter.**

" " " 500 to 800 pounds . . . . . . ⅝ " "

" " " 800 to 1800 pounds . . . . . . ¾ " "

" " above 1800 pounds . . . . ⅞ to 1 " "

The actual weights usually exceed above from two to three per cent.

WEIGHT OF BUILDINGS.

[From the "American Architect."]

It has been calculated that the pressure per square foot of the superstructure upon the foundation walls of a few of the best-known buildings is as follows:—

Dome of United-States Capitol at Washington,	13,477	pounds
Girard College, Philadelphia	13,440	"
St. Peter's, Rome	33,330	"
St. Paul's, London	39,450	"
St. Geneviève, Paris	60,000	"
Le Toussaint, Angers	90,000	"

while the pressure upon the earth per square foot in the case of St. Paul's, London, is 42,950 pounds.

COST OF PUBLIC BUILDINGS.

An experienced architect and surveyor, on the 19th of February, 1879, prepared, and presented to Gen. Meigs, Quartermaster-General, the estimate which follows of the cost of various public and private buildings in this country, the comparison being by cubic feet, external dimensions:—

Sub-Treasury and Post-Office, Boston, Mass.	$2,080,507
United-States Branch Mint, San Francisco, Cal. . . .	1,500,000
Custom and Court House and Post-Office, Cairo, Ill. .	271,081
Custom and Court House and Post-Office, Columbia, S.C.	381,900
United-States building, Des Moines, Io.	221,437
United-States building, Knoxville, Tenn.	398,847
United-States building, Madison, Wis.	329,389
United-States building, Ogdensburg, N.Y.	216,576
United-States building, Omaha, Neb.	334,000
United-States building, Portland, Me..	302,215
German Bank, Fourteenth Street, Newport, R.I. . . .	475,000
Staats-Zeitung, New-York City	475,100
Western Union Telegraph, New-York City	1,400,000
Masonic Temple, New-York City	1,900,000
Centennial building, Shepherd's, corner Twelfth and Pennsylvania Avenues, Washington, D.C.	246,073
Add to this the United-States National Museum, fire-proof building, at Washington, D.C.	250,000

THE WEAR AND TEAR OF BUILDING MATERIALS.

At the tenth annual meeting of the Fire Underwriters' Association of the North-west, held at Chicago in September, 1879, Mr. A. W. Spalding read a paper on the wear and tear of building materials, and tabulated the result of his investigations in the following form : —

Material in Building.	Frame dwelling.		Brick dwelling (shingle roof.)		Frame store.		Brick store. (shingle roof.)	
	Average life, Years.	Per cent of depreciation per annum.	Average life, Years.	Per cent of depreciation per annum.	Average life, Years.	Per cent of depreciation per annum.	Average life, Years.	Per cent of depreciation per annum.
Brick	–	–	75	1⅓	–	–	66	1½
Plastering	20	5	30	3⅓	16	6	30	3⅓
Painting, outside . .	5	20	7	14	5	20	6	16
Painting, inside . . .	7	14	7	14	5	20	6	16
Shingles	16	6	16	6	16	6	16	6
Cornice	40	2½	40	2½	30	3⅓	40	2½
Weather-boarding . .	30	3⅓	–	–	30	3⅓	–	–
Sheathing	50	2	50	2	40	2½	50	2
Flooring.	20	5	20	5	13	8	13	8
Doors, complete . . .	30	3⅓	30	3⅓	25	4	30	3⅓
Windows, complete .	30	3⅓	30	3⅓	25	4	30	3⅓
Stairs and newel . .	30	3⅓	30	3⅓	20	5	20	5
Base	40	2½	40	2½	30	3⅓	30	3⅓
Inside blinds	30	3⅓	30	3⅓	30	3⅓	30	3⅓
Building hardware . .	20	5	20	5	13	8	13	8
Piazzas and porches .	20	5	20	5	20	5	20	5
Outside blinds . . .	16	6	16	6	16	6	16	6
Sills and first-floor joists	25	4	40	2½	25	4	30	3⅓
Dimension lumber . .	50	2	75	1⅓	40	2½	66	1½

These figures represent the averages deduced from the replies made by eighty-three competent builders unconnected with fire-insurance companies, in twenty-seven cities and towns of the eleven Western States.

CAPACITY OF ROUND TANKS AND CISTERNS, IN GALLONS.

Diameter in Inches.	Height of Tank.													Diameter in Inches.
	1 in.	2 in.	3 in.	4 in.	5 in.	6 in.	7 in.	8 in.	9 in.	10 in.	11 in.	1 ft.	2 ft.	
1	0.003	0.007	0.010	0.01	0.02	0.02	0.02	0.03	0.03	0.03	0.04	0.04	0.08	1
2	0.014	0.027	0.040	0.05	0.07	0.08	0.10	0.11	0.12	0.14	0.15	0.16	0.32	2
3	0.031	0.061	0.092	0.12	0.15	0.18	0.21	0.24	0.27	0.31	0.34	0.37	0.73	3
4	0.054	0.108	0.163	0.22	0.27	0.33	0.38	0.44	0.49	0.54	0.60	0.65	1.30	4
5	0.085	0.170	0.255	0.34	0.43	0.51	0.59	0.68	0.77	0.85	0.93	1.02	2.04	5
6	0.122	0.244	0.367	0.49	0.61	0.73	0.87	0.98	1.10	1.22	1.34	1.47	2.94	6
7	0.166	0.333	0.500	0.67	0.83	1.00	1.17	1.33	1.50	1.67	1.83	2.00	4.00	7
8	0.218	0.435	0.653	0.87	1.09	1.31	1.52	1.74	1.96	2.18	2.39	2.61	5.22	8
9	0.275	0.550	0.826	1.10	1.38	1.64	1.93	2.20	2.48	2.75	3.03	3.28	6.57	9
10	0.340	0.680	1.020	1.36	1.70	2.04	2.38	2.72	3.06	3.40	3.74	4.08	8.16	10
11	0.411	0.823	1.234	1.65	2.06	2.47	2.88	3.30	3.70	4.11	4.52	4.94	9.87	11
12	0.490	0.979	1.469	1.96	2.45	2.94	3.43	3.92	4.41	4.90	5.39	5.87	11.73	12
13	0.575	1.149	1.723	2.31	2.87	3.45	4.02	4.62	5.17	5.75	6.32	6.90	13.79	13
14	0.666	1.332	1.999	2.67	3.33	4.00	4.66	5.33	6.00	6.66	7.33	8.00	15.99	14
15	0.765	1.530	2.259	3.06	3.82	4.59	5.36	6.12	6.88	7.65	8.41	9.18	18.36	15
16	0.870	1.740	2.621	3.48	4.35	5.22	6.09	6.96	7.83	8.70	9.57	10.44	20.89	16
17	0.983	1.965	2.947	3.93	4.91	5.90	6.88	7.86	8.84	9.83	10.81	11.79	23.58	17
18	1.102	2.203	3.304	4.41	5.51	6.60	9.71	8.81	9.91	11.02	12.12	13.22	26.44	18
19	1.223	2.455	3.682	4.91	6.14	7.36	8.59	9.82	11.05	12.27	13.50	14.73	29.54	19
20	1.360	2.720	4.080	5.44	6.80	8.16	9.52	10.88	12.34	13.60	14.96	16.32	32.64	20
21	1.499	2.999	4.498	6.00	7.50	9.00	10.50	11.99	13.50	14.99	16.49	17.99	35.98	21
22	1.646	3.291	4.936	6.58	8.23	9.87	11.52	13.16	14.81	16.46	18.10	19.75	39.49	22
23	1.799	3.598	5.397	7.19	8.99	10.79	12.59	14.40	16.19	17.99	19.78	21.58	43.17	23
24	1.958	3.916	5.874	7.83	9.79	11.75	13.71	15.67	17.63	19.58	21.54	23.50	47.00	24
25	2.125	4.250	6.375	8.50	10.63	12.75	14.87	17.00	19.13	21 25	23.37	25.50	51.00	25
26	2.298	4.596	6.894	9.19	11.49	13.79	16.09	18.40	20.68	22.98	25.28	27.58	55.16	26
27	2.479	4.958	7.437	9.91	12.39	14.87	17.35	19.83	22.31	24.79	27.26	29.74	59.49	27
28	2.665	5.330	7.995	10.66	13.33	15.99	18.66	21.32	23.99	26.66	29 32	31.99	63.97	28
29	2.859	5.718	8.677	11.44	14.30	17.16	20.01	22.87	25.73	28.59	31.45	34.31	68.62	29
30	3.060	6.120	9.180	12.24	15.30	18.36	21.42	24.48	27.54	30.60	33.66	36.72	73.44	30

Capacity of Round Tanks and Cisterns, in Gallons (*Concluded*).

Diameter in Inches.	Height of Tank. 3 in.	6 in.	9 in.	1 ft.	2 ft.	3 ft.	4 ft.	4 ft. 6 in.	5 ft.	5 ft. 6 in.	6 ft.	6 ft. 6 in.	7 ft.	Diameter in Inches.
31	9.7	19.4	29.1	38.8	77	116	154	173	193	212	232	251	270	31
32	10.4	20.8	31.2	41.6	83	125	166	187	208	229	250	271	292	32
33	11.1	22.2	33.3	44.4	89	133	178	200	222	244	266	288	310	33
34	11.8	23.6	35.4	47.2	94	141	188	212	235	258	282	306	330	34
35	12.5	25.0	37.5	50.0	100	150	200	225	250	275	300	325	350	35
36	13.2	26.4	39.6	52.8	105	158	210	236	263	289	316	342	369	36
37	13.9	27.8	41.7	55.6	111	167	222	250	278	306	334	362	390	37
38	14.7	29.4	44.1	58.8	117	176	234	263	293	322	352	382	412	38
39	15.5	31.0	46.5	62.0	124	186	248	279	310	341	372	403	434	39
40	16.3	32.6	48.9	65.2	130	195	260	293	325	357	390	422	455	40
41	17.1	34.2	51.3	68.4	137	205	274	308	342	376	410	444	378	41
42	18.0	36.0	54.0	72.0	144	216	288	324	360	396	432	468	504	42
43	18.9	37.8	56.7	75.6	151	227	302	340	378	416	454	492	530	43
44	19.7	39.4	59.1	78.8	158	236	316	355	394	433	472	511	551	44
45	20.6	41.2	61.8	82.4	165	247	330	371	412	453	494	535	576	45
46	21.6	43.2	64.8	86.4	173	259	346	389	432	475	518	561	604	46
47	22.5	45.0	67.5	90.0	180	270	360	405	450	495	540	585	630	47
48	23.5	47.0	70.5	94.0	188	282	376	423	470	517	564	611	658	48
49	24.5	49.0	73.5	98.0	196	294	392	441	490	539	588	637	686	49
50	25.5	51.0	76.5	102.0	204	306	408	459	510	561	612	663	714	50
51	26.5	53.0	79.5	106.0	212	318	424	477	530	583	636	689	742	51
52	27.6	55.2	82.8	110.4	221	331	442	497	552	607	662	717	772	52
53	28.6	57.2	85.8	114.4	229	343	458	515	572	629	686	743	800	53
54	29.7	59.4	89.1	118.8	238	356	476	535	594	653	712	771	830	54
55	30.8	61.6	92.4	123.2	246	369	492	554	615	677	738	800	862	55
56	32.0	64.0	96.0	128.0	256	384	512	576	640	704	768	832	896	56
57	33.1	66.2	99.3	132.4	265	397	530	596	662	728	794	860	926	57
58	34.3	68.6	102.9	137.2	274	412	548	617	686	755	824	893	962	58
59	35.5	71.0	106.5	142.0	284	426	568	639	710	781	852	923	994	59
60	36.7	73.4	110.1	146.8	294	440	588	661	734	807	880	953	1028	60

CAPACITY OF CISTERNS AND TANKS.

MBER OF BARRELS (31½ GALS.) IN CISTERNS AND TANKS.

Diameter, in Feet.								
5	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
23.3	33.6	45.7	59.7	75.5	93.2	112.8	134.3	157.6
28.0	40.3	54.8	71.7	90.6	111.9	135.4	161.1	189.1
32.7	47.0	64.0	83.6	105.7	130.6	158.0	188.0	220.6
37.3	53.7	73.1	95.5	120.9	149.2	180.5	214.8	252.1
42.0	60.4	82.2	107.4	136.0	167.9	203.1	241.7	283.7
46.7	67.1	91.4	119.4	151.1	186.5	225.7	268.6	315.2
51.3	73.9	100.5	131.3	166.2	205.1	248.2	295.4	346.7
56.0	80.6	109.7	143.2	181.3	223.8	270.8	322.3	378.2
60.7	87.3	118.8	155.2	196.4	242.4	293.4	349.1	409.7
65.3	94.0	127.9	167.1	211.5	261.1	315.9	376.0	441.3
70.0	100.7	137.1	179.0	226.6	289.8	338.5	402.8	472.8
74.7	107.4	146.2	191.0	241.7	298.4	361.1	429.7	504.3
79.3	114.1	155.4	202.9	256.8	317.0	383.6	456.6	535.8
84.0	120.9	164.5	214.8	272.0	335.7	406.2	483.4	567.3
88.7	127.6	173.6	226.8	287.0	354.3	428.8	510.3	598.0
93.3	134.3	182.8	238.7	302.1	373.0	451.3	537.1	630.4

Diameter, in Feet.								
14	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**
182.8	209.8	238.7	269.5	302.1	336.6	373.0	411.2	451.3
219.3	251.8	286.5	323.4	362.6	404.0	447.6	493.5	541.6
255.9	293.7	334.2	377.3	423.0	471.3	522.2	575.7	631.9
292.4	335.7	382.0	431.2	483.4	538.6	596.8	658.0	722.1
329.0	377.7	429.7	485.1	543.8	605.9	671.4	740.2	812.4
365.5	419.6	477.4	539.0	604.3	673.3	746.0	822.5	902.7
402.1	461.6	525.2	592.9	667.7	740.6	820.6	904.7	992.9
438.6	503.5	572.9	646.8	725.1	807.9	895.2	987.0	1083.2
475.2	545.5	620.7	700.7	785.5	875.2	969.8	1069.2	1173.5
511.8	587.5	668.2	754.6	846.0	942.6	1044.4	1151.5	1263.7
548.3	629.4	716.2	808.5	906.4	1009.9	1119.0	1233.7	1354.0
584.9	671.4	773.9	862.4	966.8	1077.2	1193.6	1315.9	1444.3
621.4	713.4	811.6	916.3	1027.2	1044.6	1268.2	1398.2	1534.5
658.0	755.3	859.4	970.2	1087.7	1211.9	1342.8	1480.4	1624.8
694.5	797.3	907.1	1024.1	1148.1	1279.2	1417.4	1562.7	1715.1
731.1	839.3	954.9	1078.0	1208.5	1346.5	1492.0	1644.9	1805.3

Diameter, in Feet.								
23	**24**	**25**	**26**	**27**	**28**	**29**	**30**	
493.3	537.1	582.8	630.4	679.8	731.1	784.2	839.3	
592.0	644.5	699.4	756.5	815.8	877.3	941.1	1007.1	
690.6	752.0	815.9	882.5	951.7	1023.5	1097.9	1175.0	
789.3	859.4	932.5	1008.6	1087.7	1169.7	1254.8	1342.8	
887.9	966.8	1049.1	1134.7	1223.6	1316.0	1411.6	1510.7	
986.6	1074.2	1165.6	1260.8	1359.6	1462.2	1568.2	1678.5	
1085.2	1181.7	1282.2	1386.8	1495.6	1608.7	1723.0	1846.4	
1183.9	1289.1	1398.7	1512.9	1631.5	1754.6	1882.2	2014.2	
1282.6	1396.5	1515.3	1639.0	1767.5	1900.8	2039.0	2182.0	
1381.2	1503.9	1631.9	1765.1	1903.4	2047.1	2195.9	2343.9	
1479.9	1611.4	1748.4	1891.1	2039.4	2193.3	2352.7	2517.8	
1578.5	1718.8	1865.0	2017.2	2175.4	2339.5	2509.6	2685.6	
1677.2	1826.2	1981.6	2143.3	2311.3	2485.7	2666.4	2853.5	
1775.9	1933.6	2098.1	2269.4	2447.3	2631.9	2823.3	3021.3	
1874.5	2041.1	2214.7	2395.4	2583.2	2778.1	2980.1	3189.2	
1973.2	2148.5	2321.2	2521.5	2719.2	2924.4	3137.0	3357.0	

tanks that are tapering, measure the diameter four-tenths from large end.

WEIGHT AND COMPOSITION OF AIR.

1 cubic foot of air at 32 degrees F., under a pressure of 14.7 pounds per square inch, weighs 0.080728 of a pound.

Therefore 1000 cubic feet = 80.728 pounds.

1	cubic foot = 1.292 ounces . . .	23 per cent oxygen. 77 per cent nitrogen.
1	cubic foot of air contains . . .	0.29716 ounce oxygen. 0.99484 ounce nitrogen. 1.29200 total weight.
1	cubic foot of air contains . . .	0.0185725 pound oxygen. 0.0621555 pound nitrogen. 0.080728 pound.
53.85	cubic feet of air contain . . .	1.000 pound oxygen. 3.347 pounds nitrogen. 4.347 pounds.

Carbonic acid $= CO_2 = 22$.

$C = 6$. $O = 8$. $O_2 = 16$. $6 + 16 = 22$.

For combustion to carbonic acid, 1 pound of coal requires $2\frac{2}{3}$ pounds of oxygen, or 143.6 cubic feet of air, supposing all of the oxygen to combine with the coal. 280 to 300 cubic feet of air per pound of coal is the usual allowance for imperfect combustion.

11.59 pounds of air for perfect combustion.

24.00 pounds of air for imperfect combustion.

COMPARISON OF THERMOMETERS.

To convert the degrees of different thermometers from one into the other, use the following formula: —

F stands for degrees of	Fahrenheit, or	212°	boiling-point.
C " "	Celsius,[1] or	100°	
R " "	Reamur, or	80°	

$F = \frac{9R}{4} + 32$, and $F = \frac{9C}{5} + 32$ for degrees above freezing-point.

$F = \frac{9R}{4} - 32$, and $F = \frac{9C}{5} - 32$ for degrees below freezing-point.

$C = \frac{5(F - 32)}{9}$, and $R = \frac{4(F - 32)}{9}$ for degrees above freezing-point.

$C = \frac{5(F + 32)}{9}$, and $R = \frac{4(F + 32)}{9}$ for degrees below freezing-point.

[1] Often called Centigrade.

Zero of Celsius or Reamur = + 32° Fahrenheit. Zero of Fahrenheit = − 17.77° C, or − 14.22° R.

1. How much is 8° Celsius above Zero in Fahrenheit?

$$F = \frac{9 \times 8}{5} = \frac{72}{5} = 14.4 + 32 = 46.4^\circ \text{ above.}$$

2. How much is 8° Celsius below Zero in Fahrenheit?

$$F = \frac{9 \times 8}{5} = \frac{72}{5} = 14.4 - 32 = 17.6^\circ \text{ above.}$$

IN CASES WHERE THE PRODUCT IS SMALLER THAN 32, IT INDICATES THAT THE DEGREE IS ABOVE ZERO OF FAHRENHEIT; SEE EXAMPLE 2.

3. How much is 19° Celsius below Zero in Fahrenheit?

$$F = \frac{9 \times 19}{5} - 32 = 34.2 - 32 = 2.2 \text{ below Fahrenheit.}$$

DIFFERENT COLORS OF IRON CAUSED BY HEAT

[Pouillet.]

C.	Fah.	Color.
210°	410°	Pale yellow.
221	430	Dull yellow.
256	493	Crimson.
261 370	502 680	Violet, purple, and dull blue; between 261° and 370° C. it passes to bright blue, to sea-green, and then disappears.
500	932	Commences to be covered with a light coating of oxide, loses a good deal of its hardness, becomes a good deal more impressible to the hammer, and can be twisted with ease.
525	977	Becomes nascent red.
700	1292	Sombre red.
800	1472	Nascent cherry.
900	1657	Cherry.
1000	1832	Bright cherry.
1100	2012	Dull orange.
1200	2192	Bright orange.
1300	2372	White.
1400	2552	Brilliant white, welding heat.
1500 1600	2732 2912	Dazzling white.

MELTING-POINT OF METALS.

Name.	Fah.	Fah.	Authority.
Platina	4593°		
Antimony	955	842	I. Lowthian Bell.
Bismuth	487	507	"
Tin (average)	475	–	
Lead "	622	620	"
Zinc	772	782	"
Cast-iron	2786	1922 to 2012, white; 2012 to 2192, gray	Pouillet.
Wrought-iron	2552	2733, welding heat.	"
Copper (average),	2174		

LINEAR EXPANSION OF METALS.

	Between 0° and 100° C.	For 1° C.	For 1° F.
Zinc	0.00294	–	–
Lead	0.00284	–	–
Tin	0.00222	–	–
Copper, yellow	0.00188	–	–
" red	0.00171	–	–
Forged iron [1]	0.00122	0.0000122	0.0000677
Steel [2]	0.00114	0.0000114	0.0000633
Cast-iron [1]	0.00111	0.0000111	0.0000616

For a change of 100° F. a bar of iron 1475 feet long will extend one foot. Similarly, a bar 100 feet long will extend 0.0678 of a foot, or 0.8136 of an inch.

According to the experiments of Dulong & Petit, we have the mean expansion of iron, copper, and platinum between 0° and 100° C., and 0° and 300° C., as below.

	From 0° to 100° C.	0° to 300° C.
Iron	0.00180	0.00146
Copper	0.00171	0.00188
Platinum	0.00884	0.00018

[1] Laplace & Lavoisier. [2] Ramsden.

The law for the expansion of iron, steel, and cast-iron at very high temperatures, according to Rinman, is as follows:—

	From 25° to 525° C., red heat, = 500° C.	For 1° C. = 1° Fah.
Iron	0.00714	0.0000143 = 0.0000080
Steel	0.01071	0.0000214 = 0.0000119
Cast-iron . . .	0.01250	0.0000250 = 0.0000139
	From 25° to 1300°, nascent white, = 1275° C.	
Iron	0.01250	0.00000981 = 0.00000545
Steel	0.01787	0.00001400 = 0.00000777
Cast-iron . . .	0.02144	0.00001680 = 0.00000933
	From 500° to 1500°, dull red to white heat, = 1000° C., difference.	
Iron	0.00535	0.00000535 = 0.0000030
Steel	0.00714	0.00000714 = 0.0000040
Cast-iron . . .	0.00893	0.00000893 = 0.0000050

RATIO OF EXPANSION IN 100 PARTS, ASSUMING FORGE-IRON TO EXPAND BETWEEN 0° AND 100° C., = 0.00122.

	From 0° to 100°.	25° to 525°.	25° to 1300°.	500° to 1500°.
Iron	100 per ct.	117 per ct.	80 per ct.	44 per ct.
Steel	93 "	175 "	114 "	58 "
Cast-iron	91 "	205 "	137 "	73 "

THE PROPERTIES OF WATER.

WATER was supposed to be an element, until Priestly, late in the eighteenth century, discovered, that, when hydrogen was burned in a glass tube, water was deposited on the sides. (It has been shown that the combustion of hydrogen requires eight parts, by weight, of oxygen; and vapor of water is the result.)

It was not, however, until Cavendish and Lavoisier investigated water that its chemical composition was determined.

The several conditions of water are usually stated as the solid, the liquid, and the gaseous. Two conditions are covered by the last term; and water should be understood as capable of existing in four different conditions, — the solid, the liquid, the vaporous, and the gaseous. At and below 32° F. water exists in the solid state, and is known as ice. According to Professor Rankine, ice at 32° has a specific gravity of 0.92. Thus a cubic foot of ice weighs 57.45 pounds.

When water passes from the solid to the liquid state, heat is required for liquefaction sufficient to elevate the temperature of one pound of water 143° F. This is termed the latent heat of liquefaction. According to M. Person the specific heat of ice is 0.504, and the latent heat of liquefaction 142.65.

From 32° to 39° the density of water increases; above the latter temperature the density diminishes.

Water is said to be at its maximum density at 39° F., and under pressure of one atmosphere weighs, according to Berzelius, 62.382 pounds per cubic foot.

Water is said to vaporize at 212° F., and pressure of one atmosphere (14.7 pounds); but Faraday has shown that vaporization occurs at all temperatures from absolute zero, and that the limit to vaporization is the disappearance of heat. Dalton obtained the following experimental results on evaporation below the boiling temperature: —

Temperature.	Rate of evaporation.	Barometer.
212	1.00	29.920
180	0.50	15.270
164	0.33	10.590
152	0.25	7.939
144	0.20	6.488
138	0.17	5.565

From this the general law is deduced, that the rate of surface evaporation is proportional to the elastic force of the vapor.

Thus, suppose two tanks of similar surface dimensions, and open to the atmosphere, one containing water maintained constantly at 212° F., and the other containing water at 144° F.

Then, for each pound of water evaporated in the last tank, five pounds will be evaporated in the first tank.

It should be understood that the law of Dalton holds good only for dry air; and when the air contains vapor having an elastic

force equal to that of the vapor of the water, the evaporation ceases.

The boiling-point of water depends upon the pressure. Thus at one atmosphere (14.7 pounds, 29.22″ barometer) the temperature of ebullition is 212°. With a partial vacuum, or absolute pressure of one pound (2.037″ of mercury), the boiling-point is 101.40 F.

Upon the other hand, if the pressure be 74.7 pounds absolute (60 pounds by the gauge), the temperature of evaporation becomes 307° F.

The vaporous condition of water is limited to saturation; that is to say, when water has been converted by heat into vapor (steam), and when this vapor has been furnished with latent heat sufficient to render it anhydrous, the vaporous condition ends, and the gaseous state begins.

Superheated steam is water in the gaseous state.

The temperature of the gaseous state of water, like that of the vaporous, depends upon the imposed pressure. Under pressure of one atmosphere, water exists in the solid state at and below 32° F.; from 32° to 212° it exists in the liquid state; at and above 212°, in the vaporous state; and above saturation, in the gaseous state.

It has been stated that water boils at 212°; but MM. Magnus and Donney have shown, that, when water is freed of air, it may be elevated in temperature to 270° before evaporation takes place.

The specific heat of water under the several conditions are as follows:—

Solid	0.504	Vaporous	0.475 to 1.000
Liquid	1.000	Gaseous	0.475

CONSUMPTION OF WATER IN CITIES.

Daily Average Number of Gallons of Water per Capita in the Cities named.[1]

Washington, D.C.	158	Jersey City, N.J.	99	Edinburgh, Scot.	38
New York	100	Buffalo, N.Y.	61	Dublin, Ireland	25
Brooklyn	50	Cleveland	40	Paris, France	28
Philadelphia	55	Columbus	30	Tours, "	22
Baltimore	40	Montreal	55	Toulouse, "	26
Chicago	75	Toronto	77	Lyons, "	20
Boston	60	London, Eng.	29	Leghorn, Italy	30
Albany, N.Y.	80	Liverpool "	23	Berlin, Prussia	20
Detroit	83	Glasgow, Scot.	50	Hamburg, "	33

[1] Including water used for manufacturing, fountains, and waste.

	Approximate roof surface.	Approximate surface per sq. inch of leader opening.	
Produce Exchange Build'g, New York	33,000 sq. ft.	140 sq. ft.	Twelve 5-inch leaders
Sloane Building, New York..	19,000 sq. ft.	240 sq. ft.	Two 6-inch leaders and one 4 inches × 6 inches.
Massachusetts Hospital Insurance Company's Building, Boston	6,000 sq. ft.	70 sq. ft.	Seven 4-inch leaders.
Hemenway Building, Boston.	4,000 sq. ft.	60 sq. ft.	Five 4-inch leaders.

ADHESIVE STRENGTH OF SULPHUR, LEAD, AND PORTLAND CEMENT FOR ANCHORING BOLTS.

The following test of these materials is reported in the *American Architect*, page 105, vol. xxiv.:

"Fourteen holes were drilled in a ledge of solid limestone, seven of them being 1⅜ inches in diameter and seven of them 1⅝ inches in diameter, all being 3½ feet deep. Seven ¾-inch and seven 1-inch bolts were prepared with thread and nut on one end and plain at the other end but ragged for a length of 3½ feet from the blank end.

"Four were anchored with sulphur, four with lead, and six with cement, mixed neat. Half of each were ¾-inch and half 1-inch bolts, and all of them were allowed to stand till the cement was two weeks old. At the expiration of this time a lever of sufficient power was rigged and all the bolts were pulled with the following result:

"*Sulphur*—Three bolts out of four developed their full strength, 16,000 and 31,000 pounds. One 1-inch bolt failed by drawing out under 12,000 pounds.

"*Lead*—Three bolts out of four developed their full strength, as above; one 1-inch bolt pulled out under 13,000 pounds.

"*Cement*—Five of the bolts out of six broke without pulling out; one 1-inch bolt began to yield in the cement at 26,000 pounds, but sustained the load a few seconds before it broke.

"While this experiment demonstrated the superiority of cement, both as to strength and ease of application, yet it did not give the strength per square inch of area. To determine this, four specimens of limestone were prepared, each 10 inches wide, 18 inches long, and 12 inches thick, two of them having 1¾-inch holes, and two of them 2¾-inch holes drilled in them. Into the small holes

1-inch bolts were cemented, one of them being perfectly plain round iron, and the other having a thread cut on the portion which was imbedded in the cement. Into the 2¾-inch holes were cemented 2-inch bolts similarly treated, and the four specimens were allowed to stand thirteen days before completing the experiment. At the end of this time they were put into a standard testing-machine and pulled. The plain 1-inch bolt began to yield at 20,000 pounds, and the threaded one at 21,000 pounds. The 2-inch plain bolt began to yield at 34,000 pounds, and the threaded one at 32,000 pounds, the strain in all cases being very slowly applied. The pump was then run at a greater speed, and the stones holding the 2-inch bolts split at 67,000 pounds in the case of the smooth one and at 50,000 pounds in the case of the threaded one.

"It is thus seen that cement is more reliable, stronger, and easier of application than either lead or sulphur, and that its resistance is from 400 to 500 pounds per square inch of surface exposed. It is also a well-ascertained fact that it preserves iron rather than corrodes it. The cement used throughout the experiment was an English Portland cement."

CO-EFFICIENT OF FRICTION.

[From "Engineering News."]

The ratio obtained by dividing the entire force of friction by the normal pressure is called the co-efficient of friction; hence we may define the unit or co-efficient of friction to be the friction due to a normal pressure of one pound.

This co-efficient is as follows: for

Iron on oak	.62
Cast-iron on oak	.49
Oak on oak, fibres parallel	.48
" " greased	.10
Cast-iron on cast-iron	.15
Wrought-iron on wrought-iron	.14
Brass on iron	.16
Brass on brass	.20
Wrought-iron on cast-iron	.19
Cast-iron on elm	.19
Soft limestone on the same	.64
Hard limestone on the same	.38
Leather belts on wooden pulleys	.47
Leather belts on cast-iron pulleys	.28
Cast-iron on cast-iron, greased	.10

Pivots or axes of wrought-iron or cast-iron, on brass or cast-iron pillows : —

1st,	When constantly supplied with oil	.05
2d,	When greased from time to time	.08
3d,	Without any application	.15

TO MAKE BLUE-PRINT COPIES OF TRACINGS.

The following directions, taken from the "Locomotive," cover the whole ground. The sensitized paper can be procured at stores where artists' materials are sold, all prepared, so that the process of preparing the paper by means of chemicals can then be omitted.

The materials required are as follows : —

1st, A board a little larger than the tracing to be copied. The drawing-board on which the drawing and tracing are made can always be used.

2d, Two or three thicknesses of flannel or other soft white cloth, which is to be smoothly tacked to the above board, to form a good smooth surface, on which to lay the sensitized paper and tracing while printing.

3d, A plate of common double-thick window-glass, of good quality, slightly larger than the tracing which it is wished to copy. The function of the glass is to keep the tracing and sensitized paper closely and smoothly pressed together while printing.

4th, The chemicals for sensitizing the paper. These consist simply of equal parts, by weight, of citrate of iron and ammonia, and red prussiate of potash. These can be obtained at any drug-store. The price should not be over eight or ten cents per ounce for each.

5th, A stone or yellow glass bottle to keep the solution of the above chemicals in. If there is but little copying to do, an ordinary glass bottle will do, and the solution made fresh whenever it is wanted for immediate use.

6th, A shallow earthen dish in which to place the solution when using it. A common dinner-plate is as good as any thing for this purpose.

7th, A brush, a soft paste-brush about four inches wide, is the best thing we know of.

8th, Plenty of cold water in which to wash the copies after they have been exposed to the sunlight. The outlet of an ordinary sink may be closed by placing a piece of paper over it with a weight on top to keep the paper down, and the sink filled with water, if the

sink is large enough to lay the copy in. If it is not, it would be better to make a water-tight box about five or six inches deep, and six inches wider and longer than the drawing to be copied.

9th, A good quality of white book-paper.

Dissolve the chemicals in cold water in the following proportions: One ounce of citrate of iron and ammonia, one ounce of red prussiate of potash, eight ounces of water. They may all be put into a bottle together, and shaken up. Ten minutes will suffice to dissolve them.

Lay a sheet of the paper to be sensitized on a smooth table or board: pour a little of the solution into the earthen dish or plate, and apply a good even coating of it to the paper with the brush: then tack the paper to a board by two adjacent corners, and set it in a dark place to dry; one hour is sufficient for the drying; then place its sensitized side up, on the board on which you have smoothly tacked the white flannel cloth; lay your tracing which you wish to copy on top of it; on top of all lay the glass plate, being careful that paper and tracing are both smooth and in perfect contact with each other, and lay the whole thing out in the sunlight. Between eleven and two o'clock in the summer-time, on a clear day, from six to ten minutes will be sufficiently long to expose it; at other seasons a longer time will be required. If your location does not admit of direct sunlight, the printing may be done in the shade, or even on a cloudy day; but from one to two hours and a half will be required for exposure. A little experience will soon enable any one to judge of the proper time for exposure on different days. After exposure, place your print in the sink or trough of water before mentioned, and wash thoroughly, letting it soak from three to five minutes. Upon immersion in the water, the drawing, hardly visible before, will appear in clear white lines on a dark-blue ground. After washing, tack up against the wall, or other convenient place, by the corners, to dry. This finishes the operation, which is very simple and thorough.

After the copy is dry, it can be written on with a common pen and a solution of common soda, which gives a white line.

MINERAL WOOL.

[Manufactured by the United States Mineral Wool Company.]

Mineral wool is the slag of blast furnaces converted into a fibrous state. The process consists in subjecting a small stream of the molten slag to the impelling force of a jet of steam or com-

pressed air, which divides it into innumerable small shot or spherules, forming a spray of spark-like objects. The threads are spun out immediately upon the detachment of the slag particles from the main body of the stream, their length and fineness being dependent upon the fluidity and composition of the material under treatment. When the slag is of the proper consistency, the spherules are small at the outset, and are to some extent absorbed into the fibre; but in no case will they disappear entirely, so that a great portion of the wool contains them, and is only separated from them by riddling. That portion of the mineral wool which is carried away from the shot by air-currents is very light (fourteen pounds per cubic foot), and forms an *extra grade;* while the balance has a working-weight of twenty-four pounds per cubic foot, and is called *ordinary* mineral wool.

The *extra* grade of mineral wool contains about ninety-three per cent of its volume of air, and the ordinary mineral wool eighty-eight per cent.

This air circulates with such difficulty that moderate thicknesses of the stuff prevent the passage of heat, and perfect insulation may be obtained at small cost.

Mineral wool is used in buildings to fill between the studs and joists, to keep out the cold in winter and heat in summer, and effectually closing up all passages in which vermin and insects generally make their homes, and fires are communicated without a possibility of arrest.

It is peculiarly adapted for deafening floors; because it is used dry. and is inelastic, and therefore does not transmit the vibrations necessary to the communication of sound.

Mineral wool is also used largely for packing around steam and hot-water pipes to prevent loss of heat before reaching the radiators.

Ordinary mineral wool weighs about 24 pounds per cubic foot, and is put up in bags containing from 60 to 90 pounds in each bag. It costs at the works, in Stanhope, N.J., 1 cent per pound, and at store in New-York City, 1¼ cents per pound.

Extra mineral wool weighs about 14 pounds per cubic foot, and is put up in bags containing from 25 to 45 pounds in each bag. It costs, at the works, 3 cents per pound, and at the store, New-York City, 3½ cents per pound.

RELATIVE HARDNESS OF WOODS.

Taking shell-bark hickory as the highest standard of our forest-trees, and calling that 100, other trees will compare with it for hardness as follows: —

Shell-bark hickory	100	Yellow oak	60
Pignut hickory	96	Hard maple	56
White oak	84	White elm	58
White ash	77	Red cedar	56
Dogwood	75	Wild cherry	55
Scrub oak	73	Yellow pine	54
White hazel	72	Chestnut	52
Apple-tree	70	Yellow poplar	51
Red oak	69	Butternut	43
White beech	65	White birch	43
Black walnut	65	White pine	30
Black birch	62		

HARD-WOOD LUMBER GRADES IN BOSTON.

[From the "North-western Lumberman," 1883.]

The Boston law for the survey of black walnut and cherry, ash, oak, poplar, and butternut, requires that the woods be divided into three grades, — number one, number two, and culls.

Number one includes all boards, plank, or joist that are free from rot and shakes, and nearly free from knots, sap, and bad taper: the knots must be small and sound, and so few that they would not cause waste for the best kind of work. A split in a board or plank, if parallel with the edge of a piece, is classed number one.

Number two includes all other descriptions, except when one-third is worthless; when a board, plank, or joist contains sap, knots, splits, or any other imperfections combined, making less than one-third of a piece unfit for good work, and only fit for ordinary purposes, it is number two: when one-third is worthless, it is a cull, or refuse. Refuse or cull hard wood includes all boards, plank, or joist that are manufactured badly, by being sawed in diamond shape, smaller in one part than in another, split at both ends, or with splits not parallel, large and bad knots, worm-holes, sap, rot, shakes, or any imperfections which would cause a piece of lumber to be one-third worthless or waste.

All hard woods are measured from six inches up; and all lumber sawed thin is inspected the same as if of proper thickness, but is classed as thin, and sold at the price of thin lumber.

There is no such thickness as $\frac{3}{4}$-inch lumber: the regular sizes are $\frac{5}{8}$, 1, $1\frac{1}{4}$, $1\frac{1}{2}$, 2, $2\frac{1}{2}$, 3, 4 inch, and up, on even inches. The regular lengths are 12, 14, and 16 feet; shorter than 12 does not command full market-price.

HORSE-POWER.

A horse can travel 400 yards at a walk in $4\frac{1}{2}$ minutes, at a trot in 2 minutes, and at a galop in 1 minute; he occupies in a stall from $3\frac{1}{2}$ to $4\frac{1}{2}$ feet front, and at a picket 3 feet by 9; and his average weight equals 1000 pounds.

A horse carrying 225 pounds can travel 25 miles in a day of 8 hours.

A draught-horse can draw 1600 pounds 23 miles a day, weight of carriage included.

In a horse-mill a horse moves at the rate of 3 feet in a second. The diameter of the track should not be less than 25 feet.

A horse-power, in machinery, is estimated at 33,000 pounds, raised 1 foot in a minute; but as a horse can exert that force but six hours a day, one machinery horse-power is equivalent to that of $4\frac{1}{2}$ horses.

The strength of a horse is equivalent to that of five men.

The daily allowance of water for a horse should be four gallons.

RULES FOR WEIGHTS OF CASTINGS.

Multiply the weight of the pattern by	12 for cast-iron, 13 " brass, 19 " lead, 12.2 " tin, 11.4 " zinc,	and the product is the weight of the casting.

Reduction for Round Cores and Core Prints.

RULE.—Multiply the square of the diameter by the length of the core in inches, and the product multiplied by 0.017 is the weight of the pine core to be deducted from the weight of the pattern.

Shrinkage in Castings.

Pattern-makers' Rule.	Cast-iron, $\frac{1}{8}$ Brass . . $\frac{3}{16}$ Lead . . $\frac{1}{8}$ Tin . . $\frac{1}{12}$ Zinc . . $\frac{3}{16}$	of an inch longer per lineal foot.

RULES FOR CALCULATING THE SPEED OF DRUMS AND PULLEYS.

The diameter of the driver being given, to find its number of revolutions.

RULE. — Multiply the diameter of the driver by the number of its revolutions, and divide the product by the diameter of the driven: the quotient will be the number of revolutions of the driven.

The diameter and revolutions of the driver being given, to find the diameter of the driven that shall make any given number of revolutions in the same time.

RULE. — Multiply the diameter of the driver by its number of revolutions, and divide the product by the number of revolutions of the driven: the quotient will be its diameter.

To ascertain the size of the driver.

RULE. — Multiply the diameter of the driven by the number of revolutions you wish it to make, and divide the product by the revolutions of the driver: the quotient will be the diameter of the driver.

N. B. — In ordering pulleys, be careful to give the exact size of the shaft on which they are to go; also state how you wish them finished on the face, —*flat* face for shifting belt, *rounding* for non-shifting belt.

WEIGHT OF GRINDSTONES.

RULE. —Square the diameter (in inches), multiply by thickness (in inches), then multiply by decimal 0.06363.

EXAMPLE. — Find the weight of a stone 4 feet 6 inches diameter and 7 inches thick.

4 feet 6 inches = 54 inches; square of 54 = 2916; multiplied by 7 = 20412; multiplied by 0.06363 = *Ans.* 1298.815 pounds, which is weight of stone.

MISCELLANEOUS MEMORANDA.

Weight of Men and Women. — The average weight of twenty thousand men and women weighed at Boston, 1864, was, — men, 141½ pounds; women, 124½ pounds.

Smallest Convenient Size of slab for a 14-inch wash-bowl, 21 by 24 inches. Height of slab from floor, 2 feet 6 inches. Very small (12-inch) corner wash-bowl; slab, 1 foot 11 inches each side.

Urinals should be 2 feet 2 inches between partitions; partitions 6 feet high.

Space occupied by Water-Closets, 2 feet 6 inches wide, 2 feet deep.

Dimensions of Double Bed. — 6 feet 6 inches by 4 feet 6 inches.

Dimensions of Single Beds (in dormitories). — 2 feet 8 inches by 6 feet 6 inches.

Dimensions of a Bureau. — 3 feet 2 inches wide, 1 foot 6 inches deep, and upwards.

Dimensions of a Washstand (common chamber-sets). — 2 feet 4 inches wide, 1 foot 6 inches deep.

Dimensions of a Barrel. — Diameter of head, 17 inches; bung, 19 inches; length, 28 inches; volume, 7680 cubic inches.

Dimensions of Billiard-Tables (Collender). — 4 feet by 8 feet, 4 feet 2 inches by 9 feet, and 5 feet by 10 feet. Size of room required, 13 feet by 17 feet, 14 feet by 18 feet, and 15 feet by 20 feet respectively.

Horse-Stalls. — Width, 3 feet 10 inches to 4 feet, or else 5 feet or over in width, 9 feet long. Width should never be between 4 and 5 feet, as in such cases the horse is liable to cast himself.

Dimensions of Drawings for Patents (United States). — 10 × 15 inches, with border line one inch inside all around.

Pitch of Tin, Copper, or Tar-and-Gravel Roof. — Five-eighths of an inch to the foot, and upwards.

A fall of one-tenth of an inch in a mile will produce a *current in rivers.*

Melted snow produces from one-fourth to one-eighth of its bulk in water.

At the depth of forty-five feet, the temperature of the earth is uniform throughout the year.

A spermaceti candle 0.85 of an inch in diameter consumes an inch in length in an hour.

Velocity of sound in water, 4708 feet per second.

Avenues of City of New York run 28° 50′ 30″ east of north.

Average Height of Hand Rail to Stairs in Dwellings. — 2 feet 7 inches from top of step on line with riser.

Dimensions of Steinway Pianos.

Grand parlor, 7 -octave,	6 ft. 0 in. × 4 ft. 8½ in., to 7 ft. 3½ in. × 4 ft. 8½ in.
Grand parlor, 7⅓-octave,	8 ft. 10 in. × 5 ft. 0 in.
Square pianos	6 ft. 8 in. × 3 ft. 4 in.
Grand square	6 ft. 11½ in. × 3 ft. 6 in.
Upright piano . . .	4 ft. 10 in. × 2 ft. 3½ in. × 4 ft. 0 in. high.
Upright grand . . .	5 ft. 1½ in. × 2 ft. 4 in. × 4 ft. 5½ in. high

Height of Blackboards in School-Houses.

Primary Schools.

Third class, chalk moulding	2 feet 1 inch from floor.
Second class, chalk moulding . . .	2 feet 2½ inches from floor.
First class, chalk moulding	2 feet 4 inches from floor.
Height of boards	5 feet, to allow for mottoes, etc., at top of board.
Grammar Schools.	
Top of stool moulding	2 feet 6 inches from floor.
Height of board	4 feet 6 inches.

The above are the heights adopted in the Boston schools.

Dimensions of Schoolrooms, Boston Schools. — The sizes of the rooms in the Boston schools, as adopted by the School Board, are: for grammar schools, 28 feet × 32 feet × 13 feet 6 inches high; for primary schools, 24 feet × 32 feet × 12 feet. This accommodates 56 scholars per room, in each grade, allowing 216 cubic feet per scholar in the grammar schools, and 165 cubic feet in the primary grade.

Dimensions and Weight of Fire-Engines. — From measurements of different fire-engines belonging to the city of Boston, it was found that the greatest length, including pole, was 22 feet 6 inches. The widths varied from 5 feet to 5 feet 11 inches, the average height being 8 feet 8 inches.

The average weight of 2[illegible] engines is 8000 pounds; the greatest weight being 9420 pounds, and the least 4780 pounds.

Dimensions and Weight of Hose Carriages. — Extreme length, with horse, 19 feet 6 inches; without horse, 17 feet 6 inches. Width, 5 feet 9 inches to 7 feet 0 inches; height, from 6 feet 8 inches to 7 feet 0 inches; average weight of 11 carriages, 2913 pounds; greatest weight, 3500; least weight, 2120.

Dimensions and Weight of Ladder Wagons. — Length of truck, 33 feet; total length, with ladders on, 45 feet; width, 6 feet 2 inches; average weight of 12 wagons, 6900 pounds; greatest weight, 8800; least, 4350.

Dimensions of Carriages.—*Covered Buggy* (*Goddard*). -Length over all, 14 feet; width, 5 feet; height, 7 feet 4 inches. Vill turn in space from 14 to 20 feet square, according to skill.

Coupé.—Length over all, 18 feet; width, 6 feet; height, 6 feet inches.

Buggy (*Piano Box*).—Length over all, 14 feet; width, 4 feet 10 ıches.

Landau.—Length over all, 19 feet 6 inches; width, 6 feet 3 ıches; height, 6 feet 3 inches; length of pole, 8 feet 0 inches.

Stanhope Gig (*2 Wheels*).—Length over all, 10 feet 6 inches; idth, 5 feet 8 inches; height, 7 feet 6 inches.

Victoria.—Length, without pole, 9 feet 6 inches; length of ɔle, 8 feet; width over all, 5 feet 4 inches.

Light Brougham.—Length, without pole or shaft, 9 feet to 11 ·et; width over all, 5 feet 4 inches; height, 6 feet 4 inches.

VEIGHT, PER FOOT, OF RAYMOND'S COMPRESSED LEAD SASH WEIGHTS.

Size.	Weight per lineal foot. Round weights.	Weight per lineal foot. Square weights.
1 inch.	3½ pounds.	4.93 pounds.
1¼ "	6 "	7.68 "
1½ "	8¼ "	10.27 "
1¾ "	11¾ "	15.08 "
2 inches.	15½ "	19.02 "
2¼ "	18½ "	24.00 "
2½ "	23 "	30.82 "
2¾ "	28.93 "	37.27 "
3 "	34.81 "	44.38 "
3¼ "	40.52 "	52.07 "
3½ "	47.26 "	60.82 "
3¾ "	54.00 "	69.33 "
4 "	61.93 "	

VEIGHT OF LUMBER PER THOUSAND (M) FEET.

BOARD MEASURE.

	Dry.	Partly seasoned.	Green.
Pine and hemlock	2500 lbs.	2700 lbs.	3000 lbs.
Norway and yellow pine . .	3000 "	4000 "	5000 lbs.
Oak and walnut	4000 "	5000 "	
Ash and maple	3500 "	4000 "	

WEIGHTS OF CORDWOOD.

	Lbs.	Carbon.		Lbs.	Carbon.
1 cord hickory . .	4468	100	1 cord Canada pine .	1870	42
1 " hard maple . .	2864	58	1 " yellow oak . .	2920	61
1 " beech . . .	3234	64	1 " white oak . .	1870	81
1 " ash	3449	79	1 " Lombardy poplar	1775	41
1 " birch . . .	2368	49			
1 " pitch-pine .	1903	43	1 " red oak . . .	3255	70

EXPLOSIVE FORCE OF VARIOUS SUBSTANCES USED FOR BLASTING, ETC.

(Builders' Guide and Price Book.—HODGSON.)

SUBSTANCES.	Heat.	Volume of gas.	Estimated explosive force.
Blasting-powder	509	0.173 litre.	88
Artillery powder	608	0.225 "	137
Sporting-powder	641	0.216 "	139
Powder, nitrate of soda for its base	764	0.248 "	190
Powder, chlorate of potash for its base	972	0.318 "	309
Gun-cotton	590	0.801 "	472
Picric acid	687	0.780 "	536
Picrate potash	578	0.585 "	680
Gun-cotton mixed with chlorate of potash	1420	0.484 "	680
Picric acid mixed with chlorate of potash	1424	0.408 "	582
Picrate mixed with chlorate of potash	1422	0.337 "	478
Nitro-glycerine	1320	0.710 "	939

The above table is by the celebrated M. Berthelot, who further describes nitro-glycerine "as really the ideal of portable force. It burns completely without residue: in fact, gives an excess of oxygen: it develops twice as much heat as powder, three and a half times more gas, and has seven times the explosive force, weight for weight, and, taken volume for volume, it possesses twelve times more energy." From the extreme danger of the work, none but a competent chemist should attempt to manufacture it.

FORCE OF THE WIND.

(Builders' Guide and Price Book.)

Miles per Hour.	Feet per minute.	Feet per second.	Force, in lbs., per square foot.	Description.
1	88	1.47	0.005	Hardly perceptible.
2	176	2.93	0.020	Just perceptible.
3	264	4.4	0.044	
4	352	5.87	0.079	Gentle breeze.
5	440	7.33	0.123	
10	880	14.67	0.492	Pleasant breeze.
15	1320	22	1.107	
20	1760	29.3	1.970	Brisk gale.
25	2200	26.6	3.067	
30	2640	44	4.429	High wind.
35	3080	51.3	6.027	
40	3520	58.6	7.870	Very high wind.
45	3960	66	9.900	
50	4400	73.3	12.304	Storm.
60	5280	88	17.733	Great storm.
70	6160	102.7	24.153	
80	7040	117.3	31.490	Hurricane.
100	8800	146.6	49.200	

MAIL CHUTES.

The Cutler Patent Mailing System, or United States Mail Chute, has now come to be very generally used in office buildings, public buildings, hotels, and apartment houses, in connection with which the United States free collection service is available. It is, therefore, important that architects should be informed with regard to the simple but necessarily rigid conditions under which this method of handling mail can be availed of.

The chute must extend in a vertical line, must be exposed to view and accessible throughout its entire length. It is made in removable sections, to facilitate clearing it in the event of accident.

The Cutler Manufacturing Company, of Rochester, N. Y., who are the owners of the original and subsequent patents under which the device is manufactured, publish this information at length, illustrated by detail drawings, which can be obtained by any architect, on application, and without charge.

REFRIGERATORS.

The following information is given as a guide to architects in providing for refrigerators in fine residences, hotels, club buildings, etc.

A consultation with some reliable refrigerator builder, however, is always wise before deciding in relation to space to be occupied by refrigerators, refrigerating rooms, freezers, etc., as *a satisfactory refrigerator cannot be adapted to a badly proportioned space.* Care should be taken to select a refrigerator simple in its working and *easily cleansed*, as modern sanitary science has traced much sickness to poor refrigeration. *Thorough insulation* is one of the most important features in a refrigerator, as upon this depends economy in the use of ice, the keeping of the cold air, and the consequent perfect preservation of the food.

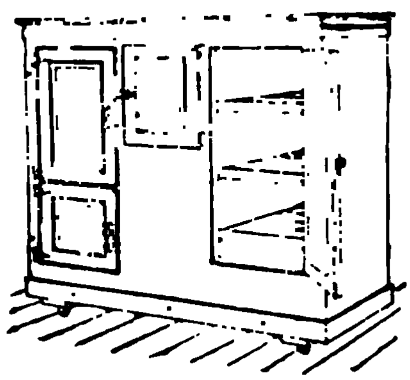


Fig. 1.

Fig. 1 is a kitchen refrigerator for use in families of ordinary size, and has the ice located in the centre. Depth should not be over three feet nor under two feet. Height may be four to seven feet. Length of front largely determines the capacity, and should be, say, from five to seven feet.

Fig. 2 shows greater capacity, and is better adapted for use in large families, entertaining considerably, and for small clubs, boarding-houses, restaurants, private hospitals, etc. This style is known as a "combination" refrigerator, from the fact that it contains separate compartments for the various kinds of food. The large compartment at the left is specially for large meats, and packages in bulk, and is fitted with shelves and meat hooks. The right end of the refrigerator is divided by a partition into two compartments, the drawers being for steaks, chops, jellies, etc., and the door above for vegetables and sundries. The compartment to the right of this is specially for milk and butter, and should be absolutely separate from all other compartments. One ice tank supplies cold air to all compartments, and is filled through a door in the front.

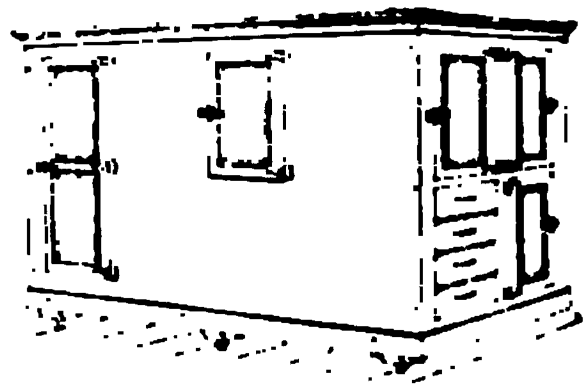


Fig. 2.

A convenient arrangement is a window in the wall at back of refrigerator, through which ice may be passed into refrigerator.

Refrigerators over two feet in depth should be built in sections

bolted together, rendering them easy to transport and handle in contracted space.

Fig. 3 is a refrigerator for use in butler's pantries, where economy of space is important. The ice tank is arranged to come out on a runway, for convenience in filling. When the ice tank is pushed back, this runway folds up, and an outside door closes over it. This does away with the necessity of cutting through the counter-top, and permits the ice tank to be readily taken out for cleansing purposes. The height should be about two feet eight inches, depth about two feet. Length of front determines capacity, but should never be less than two feet ten inches. In every three feet or three feet six inches one ice tank is allowed. The finish, wood, trim, and hardware should correspond with other fittings.

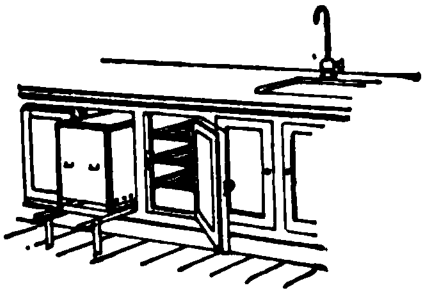


Fig. 3.

Drainage.—A *short*, *accessible*, well-trapped drain is imperative, and should be as nearly under the centre of the ice compartment as possible. It is well to have refrigerators on casters, so they are easily moved for cleaning about them.

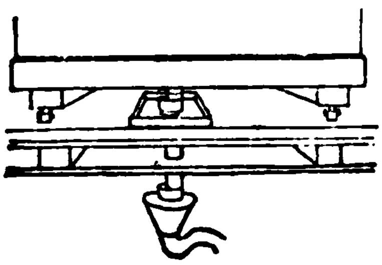


Fig. 4.

Fig. 4 shows a good drainage arrangement, permitting removal of refrigerator at will.

Plumber's pan for reception of refrigerator drip should be countersunk in floor.

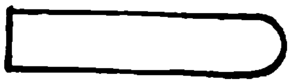
Where a very low temperature is required, as for game or fish carried in large quantities, or in medical colleges where the object is to preserve bodies, it is absolutely necessary that ice should go into the tanks from top.

Usual complement of refrigerators for use in ordinary families: one in kitchen; one in butler's pantry. Large families same, with greater capacity. Small clubs, small restaurants, etc.: one general storage; one wine; one in or near kitchen, for cook's use; one fish. Large hotels, clubs, restaurants, etc.: one storage for large meat; one in or near kitchen, for cook's use; one fish; one milk and butter; one in storeroom; one ice-cream (in hotels); one wine. Private hospitals: one large storage; one cook's use in or near kitchen; one milk and butter; one iron-lined box for broken ice. Large hospitals same, but increased capacity, and a small refrigerator in each ward. Isolated hospitals should have large storage ice-houses in addition. Medical colleges, for preserving bodies, with accommodations for eight bodies: dimensions, about 8′ 6″ front, 7′ 6″ deep, and 9′ high. Ice going into tanks from top.

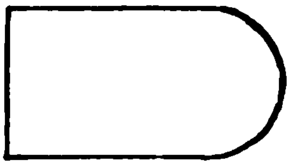
CLASSICAL MOULDINGS.

Mouldings are so called because they are of the same shape throughout their length as though the whole had been cast in the same mould or form. The regular mouldings, as found in remains of classic architecture, are eight in number, and are known by the following names :—

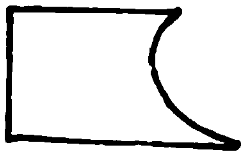
Annulet, band, cincture, fillet, listel, or square.



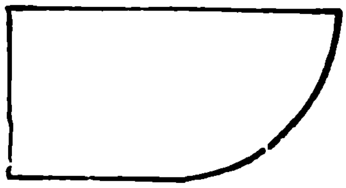
Astragal, or bead.



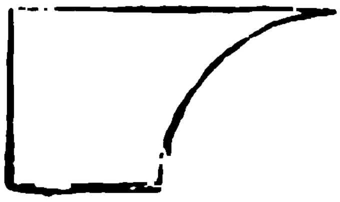
Torus, or tore.



Scotia, trochilus, or mouth.



Ovolo, quarter-round, or echinus.



Cavetto, cove, or hollow.

Cymatium, or cyma-recta.

Inverted cymatium, or cyma-reversa.

The last two are both called "ogee."

Some of these terms are derived thus : Fillet, from the French word *fil*, "thread;" astragal, from *astragalos*, "a bone of the heel," or "the curvature of the heel;" bead, because this moulding, when properly carved, resembles a string of beads; torus, or tore, the Greek for *rope*, which it resembles when on the base of a column : scotia, from *skotia*, "darkness," because of the strong shadow which its depth produces, and which is increased by the projection of the torus above it ; ovolo, from *ovum*, "an egg,"

which this member resembles, when carved, as in the Ionic capital; cavetto, from *cavus*, "hollow;" cymatium, from *kumaton*, "a wave."

Characteristics of Mouldings. — Neither of these mouldings is peculiar to any one of the orders of architecture; and although each has its appropriate use, yet it is by no means confined to any certain position in an assemblage of mouldings. The use of the fillet is to bind the parts, as also that of the astragal and torus, which resemble ropes. The ovolo and cyma-reversa are strong at their upper extremities, and are therefore used to support projecting parts above them.

The cyma-recta and cavetto, being weak at their upper extremities, are not used as supporters, but are placed uppermost to cover and shelter the upper parts. The scotia is introduced in the base of a column to separate the upper and lower torus, and to produce a pleasing variety and relief.

The form of the bead and that of the torus is the same: the reasons for giving distinct names to them are, that the torus, in every order, is always considerably larger than the bead, and is placed among the base mouldings, whereas the bead is never placed there, but on the capital or entablature. The torus, also, is seldom carved, whereas the bead is; and while the torus, among the Greeks, is frequently elliptical in its form, the bead retains its circular shape. While the scotia is the reverse of the torus, the cavetto is the reverse of the ovolo, and the cyma-recta and cyma-reversa are combinations of the ovolo and cavetto.

THE CLASSICAL ORDERS.

The term "order," in its architectural meaning, refers to the system of columniation practised by the Greeks and Romans, and is employed to denote the columns and entablature together. These two divisions combined constitute an order, and so far all orders are alike; but, as there were certain distinct styles of columns and entablatures employed by the Greeks and Romans, the orders have been divided into five classes, which are commonly known as the *Five Orders*.

The plainest and simplest of the orders is the TUSCAN Order, which was used by the early Romans, and supposed to have been borrowed by them from the Etruscans; the next three orders, viz., the DORIC, IONIC, and CORINTHIAN, were originated and perfected by the Greeks; and the last, or COMPOSITE Order, was the work of the Roman artists, who endeavored to improve upon the Greek Corinthian.

The ancient Greeks and Romans, using these orders continually, brought them to perfection; and the best examples of the different orders have in modern times served as guides in designing classical buildings.

As has been stated, an order consists of two divisions, the column and entablature; and each of these is subdivided into three distinct parts or members, — viz., the column, into *base*, *shaft*, and *capital*; the entablature, into *architrave*, *frieze*, and *cornice*.

That those who wish to employ any of the orders in their designs may readily draw them in the right proportions, the different orders have been analyzed, and a certain size given to each part in terms of the *diameter* of the column. For this purpose the lower diameter of the column is taken as the *proportional* measure for all the other parts and members of an order, for which purpose it is subdivided into sixty parts, called minutes. Being proportional measures, diameters and minutes are not fixed ones like feet and inches, but are variable as to the actual dimensions which they express, — larger or smaller, according to the actual size of the diameter of the column. For example, if the diameter be just five feet, a minute, being one-sixtieth, will be exactly one inch.

In the following engravings which are taken from Hatfield's "House Carpenter," the numbers in column H denote the height of the parts opposite them in *minutes*; and the numbers in column P denote the projection of the corresponding part from the axis of the column, also in minutes.

Some writers give the proportions of the parts in *diameters*, *modules*, and *minutes*; the module being half a diameter, or thirty minutes. Its use, however, rather complicates the measurements, instead of simplifying them.

The following definition of the five orders is taken from "The House Carpenter" (John Wiley & Sons, publishers), and corresponds with what is generally given in other architectural works.

THE TUSCAN ORDER (Fig. 1) is said to have been introduced of the Romans by the Etruscan architects, and to have been the only order used in Italy before the introduction of the Grecian orders.

It is the plainest order used by the Romans, it having but few mouldings, and no carving or enrichments. The shaft was more [illegible] than the Doric, and had a base consisting of a plinth and [illegible] forms, connected with the body of the shaft by a [illegible]. Although the capital had the same individual mouldings as the Doric, they did not project nearly as far. The use of this order [illegible] was very limited, owing to its rudeness; and all that is

known concerning it is from Vitruvius, no remains of buildings in this style being found among ancient ruins.

FIG. 1.—MODIFIED TUSCAN ORDER.

THE DORIC ORDER (Fig. 2) is the oldest and simplest of the Greek orders. Its principal features, as well as its mouldings and ornaments, are simple; its character is severe, and it bears throughout the impress of repose, solidity, and strength. The Doric columns, which are short, powerful, and closely ranged together, in order to support the weight of the massive entablature, consist of

the shaft and the capital, and rest immediately, without base, on the upper step, which serves as the ground floor of the temple.

Fig. 2.—Grecian Doric.

The shaft is channelled perpendicularly into twenty flutes, which have a sharp edge or arris; and is greatly diminished towards the top, so that the diameter above is much less than at the base.

This tapering does not take place in a straight line, but by a gradual decrease in a gentle parabolic curve, which is known as the entasis.

The architrave is a rectangular block separated by a projecting fillet from the frieze. The frieze of the Doric Order is not taken up with sculpture in uninterrupted succession; but it occurs in groups, at regular intervals, separated by features called triglyphs, which are quadrangular projecting slabs, higher than they are broad, with perpendicular channels, and are to be considered as supports of the cornice. They are distributed in such a way that one occurs over the middle of each column, and of each intervening space; in the case of the corner columns, however, the triglyphs are introduced at the corners, and not over the centre of the column. The spaces formed between the triglyphs are called metopes. They are either squares, or oblongs of greater breadth than height, and were originally open. After they were closed, alto-reliefs were generally introduced, which in the larger temples represented the deeds of gods and heroes, and in the smaller ones the skulls of animals.

The Doric was much more largely used in Italy and Sicily than either of the other orders, and in the classical buildings of modern times it is very commonly found. It is very suitable for the lower story of a façade which has two or more orders, one above the other.

THE IONIC ORDER (Fig. 3) did not come into use until the Doric had been perfected and in use for a long time. According to historians, it was invented by Hermogenes of Alabanda; and he being a native of Caria, then in the possession of the Ionians, the order was called the Ionic.

The distinguishing features of this order are the volutes or spirals of the capital, and the dentils among the bed-mouldings of the cornice; although, in some instances, dentils are wanting. The Ionic Order also has more mouldings than the Doric; its forms are richer and more elegant; and, as a style, it is lighter and more graceful than the Doric. The Doric Order has been compared to the male and the Ionic to the female figure. The Ionic column has a less diminished shaft, and a smaller parabolic curve, than the Doric. It is like the Doric, channelled; the flutings, which are twenty-four in number, are separated by annulets, and are therefore narrower but at the same time deeper than the Doric, and are terminated at the top and bottom by a final curvature.

This order differs from the Doric, also, in having a base, which is generally of the Attic form, as shown in Fig. 3.

To describe the Ionic Volute. — Draw a perpendicular from *a* to *s* (Fig. 4), and make *as* equal to 20 min., or to $\frac{4}{9}$ of the whole height *ac*; draw *so* at right angles to *sa*, and equal to $1\frac{1}{4}$ min.;

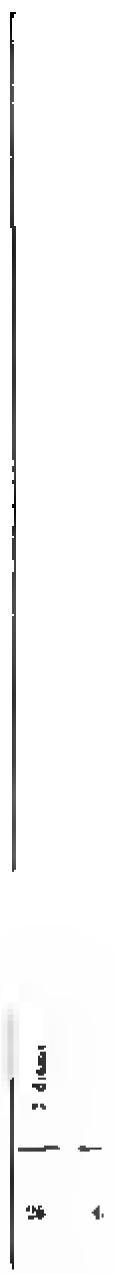


Fig. 3. — Grecian Ionic.

upon *o*, with $2\frac{1}{2}$ min. for radius, describe the eye of the volute; about *o*, the centre of the eye, draw the square *rt*12, with sides equal to half the diameter of the eye, viz., $2\frac{1}{2}$ min., and divide it

into 144 equal parts, as shown at Fig. 5. The several centres in rotation are at the angles formed by the heavy lines, as figured, 1, 2, 3, 4, 5, 6, etc. The position of these angles is determined by commencing at the point 1, and making each heavy line one part less in length than the preceding one. No. 1 is the centre for the arc *ab* (Fig. 4); 2 is the centre for the arc *bc*; and so on to the last.

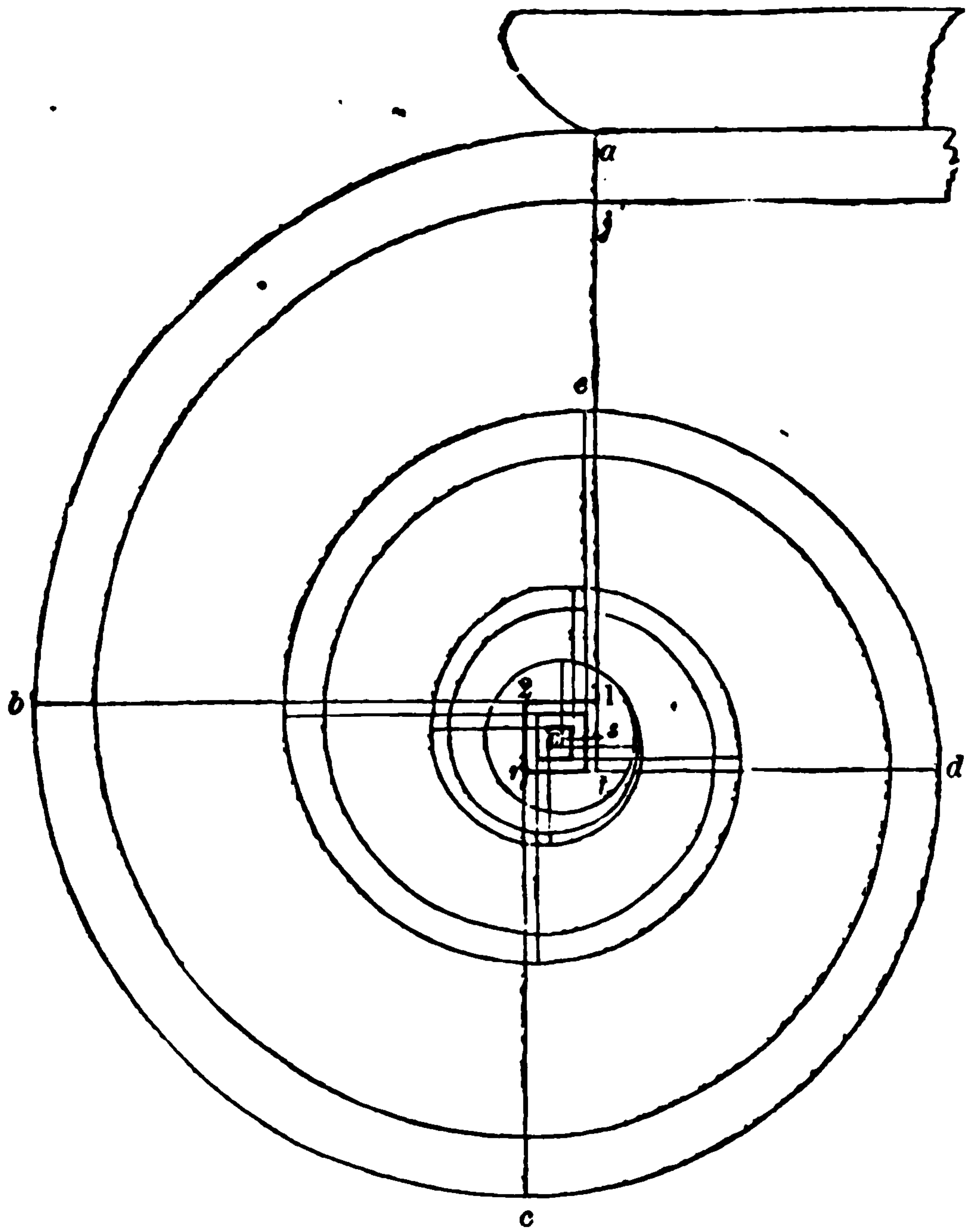


FIG. 4.—IONIC VOLUTE.

The inside spiral line is to be described from the centres *x*, *x*, *x*, etc. (Fig. 5), being the centre of the first small square towards the middle of the eye from the centre for the outside arc. The breadth of the fillet at *aj* is to be made equal to $2\frac{3}{10}$ min. This is for a spiral of *three* revolutions: but one of any number of revolutions, as 4 or 6, may be drawn, by dividing *of* (Fig. 5) into a corresponding number of equal parts. Then divide the part nearest the centre *o* into two parts, as at *h*; join *o* and 1, also *o* and 2; draw *h*3 parallel to *o*1, and *h*4 parallel to *o*2; then the lines *o*1, *o*2, *h*3, *h*4

will determine the length of the heavy lines, and the place of the centres.

FIG 5. — EYE OF VOLUTE.

THE CORINTHIAN ORDER (Fig. 6) is in general like the Ionic, though its proportions are lighter and more slender, and the individual parts are more rich and elegant. The distinguishing feature of the order is its beautiful capital, which has the shape of an expanded calyx, its form being borrowed from organic nature. The acanthus, or brank-ursine, is imitated in the leaves, as well as in the buds and stalks. The abacus is square in shape, with its sides curved into a retreating semicircle, and its truncated corners covered by the volutes shown in the engraving. The Attic base is often used with this order, the same as with the Ionic, although a different base is shown in the cut.

GRECIAN ORDERS MODIFIED BY THE ROMANS. — The orders of Greece were introduced into Rome in all their perfection. But the luxurious Romans, not satisfied with the simple elegance of their refined proportions, sought to improve upon them by lavish displays of ornament. They transformed in many instances the true elegance of the Grecian art into a gaudy splendor, better suited to their less refined taste. The Romans remodelled each of the orders. The Doric was modified by increasing the height of the column to eight diameters; by changing the echinus of the capital for an ovolo, or quarter-round, and adding an astragal and neck below it; by placing the *centre*, instead of one edge, of the first triglyph over the centre of the column; and introducing horizontal instead of inclined mutules in the cornice, and in some instances dispensing with them altogether. The Ionic was modified by diminishing the size of the volutes, and, in some specimens, introducing a new capital in which the volutes

were diagonally arranged. This new capital has been termed *modern* Ionic. The favorite order at Rome and her colonies was the Corinthian. But this order the Roman artists, in their

Fig 6. — Grecian Corinthian.

search for novelty, subjected to many alterations, especially in the foliage of its capital. Into the upper part of this they introduced the modified Ionic capital; thus combining the two in one. This change was dignified with the importance of an *order*, and received the appellation of the Composite Order, the best specimen of which is found in the Arch of Titus (Fig. 7). This style was not

much used among the Romans themselves, and is but slightly appreciated now.

FIG. 7. — COMPOSITE ORDER. — ARCH OF TITUS.

EGYPTIAN STYLE. — The architecture of the ancient Egyptians is characterized by boldness of outline, solidity, and grandeur.

The principal features of the Egyptian style of architecture are: uniformity of plan, never deviating from right lines and angles; thick walls, having the outer surface slightly deviating inwardly from the perpendicular; the whole building low; roof flat, composed of stones reaching in one piece from pier to pier, these being

supported by enormous columns, very stout in proportion to their height; the shaft sometimes polygonal, having no base, but with a great variety of handsome capitals, the foliage of these being of the palm, lotus, and other leaves; entablatures having simply an

FIG. 8.—EGYPTIAN ARCHITECTURE.

architrave, crowned with a huge cavetto ornamented with sculpture; and the intercolumniation very narrow, usually $1\frac{1}{2}$ diameters and seldom exceeding $2\frac{1}{2}$. In the remains of a temple the walls were found to be 24 feet thick; and at the gates of Thebes, the walls at the foundation were 50 feet thick, and perfectly solid. The immense stones of which these, as well as Egyptian walls

generally, were built, had both their inside and outside surfaces faced, and the joints throughout the body of the wall as perfectly close as upon the outer surface.

The dimensions and extent of the buildings may be judged from the Temple of Jupiter at Thebes, which was 1400 feet long and 300 feet wide, exclusive of the porticos of which there was a great number.

A great dissimilarity exists in the proportion, form, and general features of Egyptian columns. For practical use the column shown in Fig. 8 may be taken as a standard of the Egyptian style.

LIST OF NOTED ARCHITECTS.

[GWILT.]

BEFORE CHRIST.

NAME OF ARCHITECT.	Century.	Principal works.
Theodorus, of Samos.	7th	Labyrinth at Lemnos, some buildings at Sparta, and the Temple of Jupiter at Samos.
Ictinus, of Athens.	6th	Parthenon at Athens, Temple of Ceres and Proserpine at Eleusis, Temple of Apollo Epicurius in Arcadia.

Before Christ.

ME OF ARCHITECT.	Century.	Principal works.
ates, of Athens.	6th	Assisted Ictinus in the erection of the Parthenon.
:les, of Athens.	6th	Propylæa of the Parthenon.
'ates, of Macedonia.	4th	Rebuilt the Temple of Diana at Ephesus, engaged on works at Alexandria, was the author of the proposition to transform Mount Athos into a colossal figure.
nicus, of Athens.	4th	Tower of the Winds at Athens.
achus, of Corinth.	4th	Reputed inventor of the Corinthian order.
tus, of Cnidus.	4th	The Pharos of Alexandria.
ius, of Rome.	2d	Design for the Temple of Jupiter Olympus at Athens.
odorus, of Salamis.	2d	Temple of Jupitor Stator in the Forum at Rome, Temple of Mars in the Circus Flaminius.
lus, of Rome.	1st	Several buildings at Rome; the first Roman who wrote on architecture.

After Christ.

ME OF ARCHITECT.	Century.	Principal works.
vius Pollio, of Fano.	1st	Basilica Justitiæ at Fano; a great writer on architecture.
lorus, of Persia.	4th	Many buildings in India, and some at Constantinople; the first-known Christian architect.
us, of Padua.	5th	Assisted in the erection of the celebrated rotunda at Ravenna, the cupola of which is said to have been of one stone, thirty-eight feet in diameter and fifteen feet thick.
mius, of Trales, of ia.	6th	St. Sophia, at Constantinople.
›hus, Abbot of Peterough, afterwards made 10p of Lichfield, of land.	7th	Built the Monastery of Medeshampstede, afterwards called Peterborough.
t, Archbishop of York, England.	8th	Rebuilt York Cathedral.
aldus, of France.	9th	The Cathedral of Rheims, the earliest example of Gothic architecture.

AFTER CHRIST.

NAME OF ARCHITECT.	Century.	Principal works.
Buschetto, of Dulichium.	10th	The Cathedral or Duomo of Pisa, the earliest example of the Lombard ecclesiastical style of architecture. It was built in 1016.
Pietro di Ustamber, of Spain.	10th	Cathedral of Chatres.
Lanfranc, Archbishop of Canterbury, of England.	10th	Choir of Canterbury Cathedral, burnt in 1174.
Remigius, Bishop of Lincoln, of England.	11th	Part of Lincoln Cathedral.
Walkelyn, Bishop of Winchester, of England.	11th	Said to have erected the oldest part of Winchester Cathedral.
Mauritius, Bishop of London, of England.	12th	Built old St. Paul's, in 1033.
Alexander, Bishop of Lincoln, of England.	12th	Rebuilt Lincoln Cathedral.
Dioti Salvi, of Italy.	12th	Baptistery of Pisa, near the Campo Santo. His works were in the Lombard style, and were overloaded with minute ornaments.
Buono, of Venice.	12th	The Tower of St. Mark at Venice, which is three hundred and thirty feet high and forty feet square, built in 1154: a design for enlarging the Church of Santa Maria Maggiore, at Florence, of which the master-walls still exist; the Vicaria and the Castello del' Novo, at Naples; Church of St. Andrew, at Pistola; la Casa della Citta; Campanile at Arezzo.
Wilhelm or Guglielmo, of Germany.	12th	The Leaning Tower at Pisa, built in 1174. Bonnano and Tomaso, two sculptors of Pisa, were also engaged upon it.
William, of Sens, of England.	12th	Canterbury Cathedral.
Peter, of Colechurch, of England.	13th	Began London Bridge.
Robert, of Lusarches, of France.	13th	Cathedral of Amiens, which was continued by Thomas de Cormont, and finished by his son Renauld.
Poore, Bishop of Salisbury, of England.	13th	Began Salisbury Cathedral.
Pietro Perez, of Spain.	13th	The Cathedral of Toledo.
Robert de Coucy, of France.	13th	Rebuilt the Cathedral at Rheims.
Juan Ran, of France.	14th	Finished the building of the Church of Notre Dame, of Paris.

After Christ.

Name of Architect.	Century.	Principal works.
Rafaelle d'Urbino, of Urbino.	16th	Continued the erection of St. Peter's at Rome after the death of Bramante, his master in architecture; engaged on the buildings of the Farnese Palace; Church of Santa Maria, in Navicella, repaired and altered; stables of Agostino, near the Palazzo Farnese; Palazzo Caffarelli, now Stoppani; the gardens of the Vatican; the façade of the Church of San Lorenzo, and of the Palazzo Uggoccioni, now Pandolfini, at Florence.
Bolton, W., Prior of St. Bartholomew's, of England.	16th	Supposed to have designed Henry VII.'s Chapel, where he was master of the works.
Giovanni Gil de Hontanon, of Spain.	16th	Plan of the Cathedral of Salamanca, etc.
Michael Angelo di Buonarotti, of Florence.	16th	Library of the Medici, generally called the Laurentian Library, at Florence; model for the façade of the Church of San Lorenzo, commonly called the Capella dei Depositi; Church San Giovanni, which he did not finish; fortifications at Florence and at Monte San Miniato; monument of Julius II., in the Church of San Pietro in Vincoli, at Rome; plan of the Campidoglio, Palace of the Conservatori, building in the centre, and the flight of steps in the Campidoglio, or Capitol, at Rome; continuation of the Palace Farnese and several gates at Rome, particularly the Porta Nomentana or Pia; steeple of St. Michaele, at Ostia; the gate to the Vineyard del Patriarea Grimani; Tower of S. Lorenzo, at Ardea; Church of Santa Maria, in the Certosa, at Rome; many plans of palaces, churches, and chapels. He was employed on St. Peter's after the death of San Sallo.
Martino de Gainza, of Spain.	16th	The Chapel Royal at Seville.
Machuca, of Spain.	16th	Royal Palace of Granada.
Theodore Havens, of England.	16th	Caius College, Cambridge. A good specimen of the architecture of the day.

After Christ.

Name of Architect.	Century.	Principal works.
Carlo Maderno, of Lombardy.	16th	Altered Michael Angelo's design for St. Peter's at Rome from a Greek to a Latin cross; began the palace of Urban VIII.
Sir H. Watton, of England.	17th	Author of "The Elements of Architecture," published in London in 1624.
Inigo Jones, of England.	17th	Banqueting House; chapel, Lincoln's Inn; Surgeons' Hall; arcade, Convent Garden, London; and a vast number of other important works.
Claude Perrault, of France	17th	Façade of the Louvre, Chapel of Sceaux, Chapel of Notre Dame in the Church of the Petits Pères.
Sir Christopher Wren, of England.	17th	St. Paul's; planned the city of London after the fire, nearly all the churches therein, Hampton Court, etc.
Jules Hardouin Mansard, of France.	17th	The dome of the Hôtel des Invalides, Gallerie du Palais Royal, the Place de Louis de Grand, that des Victoires, etc. He was the nephew of François Mansard, the reputed inventor of the Mansard roof.
Alexander Jean Baptiste le Blond, of France.	18th	L'Hôtel de Vendôme, in the Rue d'Enfer, at Paris. He was employed much in Russia by Peter the Great.
Galli da Bibbiena, of Italy.	18th	Theatre at Verona, theatre at Vienna; author of two books on architecture.
James Gibbs, of Scotland.	18th	Radcliffe's Library, Oxford; the new church in the Strand; St. Martin's-in-the-Fields; King's College, Royal Library, and Senate House, Cambridge.
Sir William Chambers, of England.	18th	Somerset House and many other works, author of a treatise on civil architecture.
Robert Adam, of Scotland.	18th	Architect to George III.; author of a work on the ruins of Spalatro. His principal works are the Register Office at Edinburgh, infirmary at Glasgow, the Edinburgh University, Lansdowne House, Adelphi Terrace.
Sir John Soane, of England.	18th	Bank of England, Board of Trade, State Paper Office.
Charles Percier, of France.	18th	Architect of the Tuileries, restorations, etc., at Louvre and Tuileries.

After Christ.

Name of Architect.	Century.	Principal work.
James Essex, of England.	18th	The earliest, in modern times, who practised solely mediæval art; restoration of Ely and other cathedrals; alterations at various colleges at Cambridge and Oxford.
James Wyatt, of England.	18th	The Pantheon Assembly rooms, palace at Kew, Fonthill Abbey, Doddington Hall, Ashridge House, and many restorations.
Augustus Pugin, of England.	18th	Published "Specimens of Gothic Architecture," "Examples of Gothic Architecture," "Antiquities of Normandy," and other works.
John Nash, of England.	19th	Brighton Pavilion, Haymarket Theatre, Buckingham Palace, Regent's Park and its terraces of dwellings, Regent Street and the Quadrant improvements.
Thomas Rickman, of England.	19th	New court of St. John's College, Cambridge; restoration of the Bishop of Carlisle's palace, Cumberland; upwards of twenty-five churches in the midland counties, several private dwellings. Published "Attempt to discriminate the Styles of Architecture in England."
Carl Friedrich Schinkel, of Prussia.	19th	Hauptwache Theatre and Museum, Werder-Kirche (Gothic), Bauschule and Observatory at Berlin, theatre at Hamburg, Schloss Krzescowice, Charlottenhof, and the Nicolai-Kirche at Potsdam. Published his designs, many of which were not executed.
Guillaume Abel Blouet, of France.	19th	Published supplement to Rondelet's "L'Art de Bâtir," and revised the tenth edition of that work.
Ernst Friedrich Zwirner, of Prussia.	19th	Restoration of Cologne Cathedral church at Remagen.
David Hamilton, of Scotland.	19th	The Nelson Monument, the Royal Exchange, the Western Club-house, and other buildings at Glasgow; Hamilton Palace and Lennox Castle, Scotland.
Mr. Joseph Gwilt.	19th	Compiler of the "Encyclopædia of Architecture."

AFTER CHRIST.

NAME OF ARCHITECT.	Century.	Principal works.
James Fergusson, d. Jan., 1886.	19th	Author of the "History of Architecture."
John Henry Parker, b. in London, 1806; d. 1804.	19th	Author of the "Glossary of Architecture," "The Domestic Architecture of the Middle Ages," a revised edition of Rickman's "Gothic Architecture."
George Edmund Street.	19th	The Law Courts, London.
William Burges.	19th	Cork Cathedral, Restoration of Cardiff Castle.
Sir Gilbert Scott.	19th	Hamburg Cathedral, Edinburgh Cathedral, the Albert Memorial, Midland Station and Hotel at St. Pancras, England.

LIST OF NOTED AMERICAN ARCHITECTS.

JOHN HAVILAND, b. 1792, d. 1825; practised in .

Principal works: Pittsburgh Penitentiary; Eastern Penitentiary at Cherry Hill: Hall of Justice, New York; Naval Asylum, Norfolk; New Jersey State Penitentiary; and many other jails, asylums, and public halls.

JONATHAN PRESTON, b. 1801, d. July, 1884; practised in Boston, Mass.

Principal works: The first building of the Massachusetts Institute of Technology, and the building of the Boston Society of Natural History.

WILLIAM WASHBURN, b. in Lyme, N. H., 1808, d. in Boston, November 8, 1890; practised in Boston.

Principal works: The Fifth Avenue and Victoria Hotels in New York, and the Parker House, Tremont House, Revere House, Adams House, Young's Hotel, and the American House in Boston; the Tremont Temple, Boston; Charlestown City Hall, and many other public and private buildings.

THOMAS USTICK WALTER, LL.D., b. 1804, d. October 30, 1887; practised in Philadelphia, Pa.: was one of the original members of the American Institute of Architects, and president from : received the degree of LL.D. from Harvard Uni-

versity, being the first architect to receive that degree in this country.

Principal works: The five original buildings of Girard College, designed in 1833 and completed in 1847. Extension of the National Capitol, 1851–65; also the extensions of the Patent Office, Treasury and Post Office buildings, the dome on the old Capitol, the Congressional Library, and the Government Hospital for the Insane; also numerous other buildings of lesser importance. Mr. Walter was a member of the Franklin Institute, and of many literary and scientific associations.

ARTHUR GILMAN, b. , d. ; practised in New York and Boston, in partnership with Mr. Bryant.

Principal works: Boston City Hall; First Church, on Arlington Street, Boston, and numerous dwelling-houses in New York and Boston. In association with Mr. Edward Kendall, designed the Equitable Life Assurance Company's building on Broadway, New York.

R. G. HATFIELD, b. in Elizabeth, N. J., 1815, d. February, 1879; author of the *American House Carpenter* and *Transverse Strains*; associated for thirty-five years with his brother Oliver P. Hatfield. The firm became widely known as experts and consulting architects in matters pertaining to building construction.

Principal works: House of Refuge, Randall's Island, N. Y.; Westchester County Buildings, White Plains, N. Y.; New York Institution for the Deaf and Dumb, Seaman's Bank for Savings, City Bank building, Security Insurance Co. Building, all of New York City.

OLIVER P HATFIELD, b. , d. April, 1891.

JOHN MCARTHUR, Jr., b. in Scotland in 1823, d. January, 1890; practised in Philadelphia, Pa.

Principal works: House of Refuge, Continental Hotel, Girard House, Public Ledger Building, First National Bank Building, the Assembly Building the Broad Street Presbyterian Church, all of Philadelphia; and the Philadelphia City Hall. Also the Hospital for the Insane, at Warren, Pa; Lafayette College, Easton, Pa.; and numerous other public and private buildings in Pennsylvania and other States. Was twice tendered the position of Supervising Architect to the United States Government, but declined.

EBENEZER L. ROBERT, b. 1825, d. ; practised in New York City.

Principal works: Standard Oil Company's Building, on Broadway; the Ninth National Bank; the Baptist Church of the Epiphany, on Madison Avenue; St Paul's Methodist Church, on Fourth

Avenue, all of New York City ; and the Phœnix Insurance Company's Building, Brooklyn, N. Y.

ALEXANDER R. ESTY, b. 1827, d. July 2, 1881 ; practised in Boston. Principal works: Union Congregational Church, Boston ; Harvard Street Baptist Church, Cambridge, Mass. ; Grace Church, Newton, Mass. ; Emanuel Church, on Newbury Street, Boston : Buildings of the Colby University, Waterville, Me. ; Massachusetts State Normal Schools, at Framingham and Worcester, and the University of Rochester, N. Y.

CARL PFEIFFER, b. in Germany, d. May, 1888 ; practised in New York City.

Principal works : Fifth Avenue Presbyterian Church, New York ; Fifth Avenue Riding School, New York ; and many private houses, apartment houses, hotels, etc.

CHARLES DEXTER GAMBRILL, b. 1832, d. September 13, 1880 ; practised in New York, first in partnership with Mr. George B. Post, later with H. H. Richardson.

JOHN H. STURGIS, b. , d. ; practised in Boston, Mass., with Mr. Charles Brigham, as Sturgis & Brigham.

Principal works : Boston Museum of Fine Arts, building of the Boston Young Men's Christian Association, Church of the Advent, residence of Mr. F. L. Ames, and many other fine residences in Boston and vicinity.

A. B. MULLETT, b. 1834, d. October 20, 1890 ; supervising architect to the Treasury from to

Also engineer of the District of Columbia for several years. The Post Office buildings in New York, Boston, Cincinnati, St. Louis, and Chicago were designed by him, and also the State, War, and Navy Buildings in Washington.

HENRY HOBSON RICHARDSON, b. in Louisiana in 1838 or 1839, d. in Brookline, Mass., April, 1886. Graduated at Harvard University in 1859, studied seven years at the Ecole des Beaux-Arts in Paris. Was associated for a short time with Charles D. Gambrill of New York.

Following is a list of the works executed by him, arranged in chronological order :

1. Grace Church, Medford, Mass.
2. Boston & Albany R.R. offices, Springfield, Mass.
3. Church of the Unity, Springfield, Mass.
4. The Agawam Bank, Springfield, Mass.
5. House for William Dorsheimer, Esq., Buffalo, N. Y.
6. The State Asylum for the Insane, Buffalo, N. Y.
7. Exhibition Building, Cordova, Argentine Republic.

8. American Express Company Building, Chicago, Ill.
9. Brattle Street Church, Boston, Mass.
10. Worcester High School.
11. The Hampden County Court House, Springfield, Mass.
12. Trinity Church, Boston, Mass.
13. Cheney Buildings, Hartford, Conn.
14. Phœnix Insurance Building, Hartford, Conn.
15. House for B. W. Crowninshield, Boston, Mass.
16. The North Church, Springfield, Mass.
17. William Watt Sherman's house, Newport, R. I.
18. Portions of the New York State Capitol, Albany, N. Y.
19. Public Library, Woburn, Mass.
20. Ames Memorial Library, North Easton, Mass.
21. Sever Hall, Cambridge, Mass.
22. Ames Memorial Town Hall, Easton, Mass.
23. Trinity Church Rectory, Boston, Mass.
24. Monument to Oliver and Oakes Ames, Sherman, Wyo.
25. Gate lodge for F. L. Ames, North Easton, Mass.
26. Crane Memorial Library, Quincy, Mass.
27. Bridges for the Back Bay Park, Boston, Mass.
28. City Hall, Albany, N. Y.
29. Depot for the Boston & Albany R.R., Auburndale, Mass.
30. New Law School, Cambridge, Mass.
31. House for F. L. Higginson, Esq., Beacon Street, Boston, Mass.
32. House for General N. L. Anderson, Washington, D. C.
33. Railroad depot, Holyoke, Mass., for Conn. River R.R.
34. Depot, Palmer, Mass., for Boston & Albany R.R.
35. Depot, North Easton, Mass., Boston & Albany R.R.
36. Dairy Building, North Easton, Mass.
37. House for Grange Sard, Esq., Albany, N. Y.
38. Store on Kingston and Bedford Streets, Boston, for F. L. Ames, Esq.; also store on Washington Street.
39. Billings Library for University of Vermont, Burlington, Vt.
40. Depot, Chestnut Hill, Mass., Boston & Albany R.R.
41. Converse Memorial Library, Malden, Mass.
42. Baptist Church, Newton, Mass.
43. House for Henry Adams, Esq., Washington, D. C.
44. House for John Hay, Esq., Washington, D. C.
45. Allegheny County Buildings, consisting of Court House and Jail, Pittsburgh, Pa.
46. Wholesale warehouse for Marshall, Field & Co., Chicago, Ill.
47. Armory, Detroit, Mich.

48. Chamber of Commerce, Cincinnati, O.
49. Dwelling-house for J. J. Glessner, Esq., Chicago, Ill.
50. Dwelling-house for B. H. Warder, Esq., Washington, D. C.
51. Dwelling-house for J. J. Glessner, Esq., Chicago, Ill.
52. Dwelling-house for Robert Treat Paine, Esq., Waltham, Mass.
53. Dwelling-house for Prof. E. W. Gurney, Beverly, Mass.
54. Dwelling-house for J. R. Lionberger, Esq., St. Louis, Mo.
55. Dwelling-house for William H. Gratwick, Esq., Buffalo, N. Y.
56. Store on Harrison Avenue, Boston, for F. L. Ames, Esq.
57. Railroad depot, New London, Conn.
58. House for Prof. Hubert Herkomer, A. R. A., England.

THOMAS WISEDELL, b. in England in 1846, d. in New York, July 31, 1884. Educated in the office of Mr. R. J. Withers of London. Associated with Mr. Kimball of New York.

Principal works: Madison Square Theatre, and the "Casino," both in New York City.

JOHN WELLBORN ROOT, b. in Georgia, January 10, 1850, d. in Chicago, Ill., January 15, 1891. Entered into partnership with Daniel H. Burnham in 1873, which continued until his death. Mr. Root was the designer of the firm. They designed and executed seventy-seven public buildings, many of them of the first class, and one hundred and twenty residences. Of their public buildings the following were perhaps the most important:

Calumet Club House, Art Institute, Academy of Fine Arts, Montauk Block, Calumet Building, Rialto Office Building, Insurance Exchange Building, Grannis Block, Phœnix Building, The Rookery, Masonic Building, Woman's Temple, First Regiment Armory, all of Chicago; the Mills Block, San Francisco; Midland Hotel, Board of Trade Building, American National Bank Building, of Kansas City. Mr. Root was secretary of the American Institute of Architects at the time of his death.

HENRY O. AVERY, b. , d. 1890; studied at the School of Fine Arts, in Paris. Took an important part in designing the houses of W. K. Vanderbilt and Henry G. Marquand: a prominent member of the Architectural League of New York, the Archæological Institute, and the Society of American Artists.

HERBERT C. BURDETT, b. in Boston, 1855, d. in Buffalo, April 10, 1891; associated with J. Herbert Marling, as Marling & Burdette, and practised in Buffalo, N. Y.

Principal works: The Saturn Club House, and numerous fine residences in Buffalo.

JOSEPH MORRILL WELLS, b. 1853, d. in New York, February, 1890. Mr. Wells was a junior partner in the firm of McKim, Mead & White, architects, of New York. The movement of American architects towards the Italian Renaissance, which commenced about the year 1889, was undoubtedly caused more by his influence than that of any other single individual. Among the buildings of the firm, more especially designed by him, are: the Villard Houses on Madison Avenue, New York; the "Memorial Building" in New Britain, Conn.; façade of the Century Club, New York, and a fountain in Portland, Oregon.

THE TWENTY BEST BUILDINGS, ARCHITECTURALLY, IN THE UNITED STATES.

Out of seventy-five votes sent to the *American Architect* in 1885 for the ten best buildings in the United States, the following twenty buildings received the highest number of votes, in the order named:

1. Trinity Church, Boston; Messrs. Gambrill & Richardson Architects.
2. United States Capitol, Washington, D. C. (See page 753.)
3. House of W. K. Vanderbilt, New York; R. M. Hunt, Architect.
4. Trinity Church, New York; Mr. Richard Upjohn, Architect.
5. Jefferson Market Court House, New York; Mr. F. C. Withers, Architect.
6. State Capitol, Hartford, Conn.; Mr. R. M. Upjohn, Architect.
7. City Hall, Albany, N. Y.; Mr. H. H. Richardson, Architect.
8. Sever Hall, Cambridge, Mass.; Mr. H H. Richardson, Architect.
9. State Capitol, Albany, N. Y.; Messrs. (Fuller) Eidlitz & Richardson, Architects.
10. Town Hall, North Easton, Mass.; Mr. H. H. Richardson, Architect.
11. New City Hall, Philadelphia, Pa.; Mr. J. McArthur, Jr., Architect.
12. Casino Theatre, New York; Messrs. Kimball & Wisedell, Architects.
13. Lenox Library, New York; Mr. R. M. Hunt, Architect.
14. Produce Exchange, New York; Mr. G. B Post, Architect.
15. Columbia College, New York; Mr. C. C. Haight, Architect.
16. Broad Street R.R. Station, Philadelphia, Pa.; Messrs. Wilson Bros. & Co., Architects.
17. Crane Memorial Library, Quincy, Mass.; Mr. H H. Richardson, Architect.
18. Court House, Providence, R. I.; Messrs. Stone & Carpenter, Architects.
19. Central R.R. Station, Providence, R. I.; Mr. T. A. Tefft, Architect.
20. Harvard Memorial Hall, Cambridge, Mass.; Messrs. Ware & Van Brunt, Architects.

ARCHITECTS OF NOTED PUBLIC AND PRIVATE BUILDINGS IN THE UNITED STATES.

BUILDINGS ARRANGED ACCORDING TO LOCATION.

GOVERNMENT BUILDINGS.

United States Capitol, Washington, D. C. .... Messrs. Hallet, Hadfield, Hoban, Latrobe, Bulfinch, Walter, and Clark, Architects.

National Museum, Washington, D. C. .... Cluss & Schulye, Architects.

State, War and Navy Building, Washington, D. C. .... A. B. Mullett, Architect.

Treasury Building, Washington, D. C. .... Robert Mills, T. U. Walter, Young, Rogers, and A. B. Mullett, Architects.

United States Post Offices and Court Houses:—

Baltimore, Md .... James G. Hill, Architect.
Boston, Mass. .... A. B. Mullett, Architect.
Chicago, Ill. .... A. B. Mullett, Architect.
Cincinnati, O .... A. B. Mullett, Architect.
Detroit, Mich. .... M. E. Bell, Architect.
Kansas City, Mo. .... James G. Hill, Architect.
New York, N. Y. .... A. B. Mullett, Architect.
St. Louis, Mo. .... A. B. Mullett, Architect.

STATE CAPITOLS.

Capitol of—

Colorado, at Denver ...... E. E. Meyers & Son, Architects.
Connecticut, at Hartford.... R. M. Upjohn, Architect.
Illinois, at Springfield ..... A. H. Piquenard, Architect.
Indiana, at Indianapolis..... Edwin May, Architect.
Iowa, at Des Moines .... A. H. Piquenard, Architect.
Georgia, at Atlanta .... W. J. Edbrook and F. P. Burnham, Architects.
Louisiana, at Baton Rouge.. W. A. Freret, Architect.
Maine, at Augusta .... Charles Bulfinch, Architect.
Massachusetts, at Boston.... Charles Bulfinch; Brigham & Spofford, Architects.
Michigan, at Lansing....... E. E. Meyers, Architect.

Capitol of—
New York, at Albany.......Messrs. Fuller, Eidlitz, and H. H Richardson. Architects.
Ohio, at Columbus..........Henry & Wm. Walter, Architects.
Rhode Island, at Newport..James Munday, Architect.
Tennessee, at Nashville.....John Strickland, Architect.
Texas, at Austin .... .E. E. Meyers & Son, Architects.
Virginia, at Richmond......Thomas Jefferson.

COUNTY BUILDINGS.

Suffolk County Court House,
Boston, Mass..................Geo. A. Clough, Architect.
Cook County Court House,
Chicago, Ill ....J. J. Egan, Architect.
Arapahoe County Court House,
Denver, Col.. ... .E. E. Meyers & Son, Architects.
Jefferson Market Court House,
New York ....F. C. Withers, Architect.
Allegheny County Court House and Jail, Pittsburgh, Pa......H. H. Richardson, Architect.
Court House, Providence, R. I...Stone & Carpenter, Architects.

CITY AND TOWN HALLS.

City Hall—
Albany, N. Y..............H. H. Richardson, Architect.
Boston, Mass.. ....Gilman & Bryant, Architects.
New York, N. Y. (1803-12)..John McComb, Architect.
(New) Philadelphia, Pa......John McArthur, Jr. Architect.
Town Hall, North Easton, Mass., H. H. Richardson, Architect.

CHURCHES, ETC.

All Saints Cathedral, Albany, N. Y ....R. W. Gibson, Architect.
St. Peter's Episcopal Church, Albany, N. Y................R. M. Upjohn, Architect.
First M. E. Church, Baltimore, Md.........................McKim, Mead & White, Architects.
Brattle Street Church, Boston, Mass. ....Gambrill & Richardson, Architects.
Church of the Advent, Boston, Mass.......................Sturgis & Brigham, Architects.

First Church, Arlington Street, Boston, Mass .... Gilman & Bryant, Architects.
First Presbyterian Church, Boston, Mass .... R. M. Upjohn, Architect.
Spiritual Temple, Boston, Mass.. Hartwell & Richardson, Architects.
Trinity Church, Boston, Mass... Gambrill & Richardson, Architects.
The (New) Old South Church, Boston, Mass................. Cummings & Sears, Architects.
Emanuel Baptist Church, Brooklyn, N. Y............. ...... Francis H. Kimball, Architect.
Jewish Synagogue, New York State, Buffalo (Temple Beth Zion).............. .... E. A. & W. W. Kent, Architects.
Calvary Presbyterian Church, Cleveland, O.... .. .. C. F. Schweinfurth, Architect.
St. Stephen's Church, Lynn, Mass., Ware & Van Brunt, Architects.
Fifth Avenue Presbyterian Church, New York .... Carl Pfeiffer, Architect.
Jewish Synagogue, New York.... L. Eidlitz, Architect.
Madison Avenue M. E. Church, New York .... R. H. Robinson, Architect.
St. Patrick's R. C. Cathedral, New York .... Renwick & Sands, Architects.
Trinity Church, New York...... Richard Upjohn, Architect.
Park Avenue M. E. Church, Philadelphia. .... Hazelhurst & Huckel, Architects.
Church of the Messiah, St. Louis, Mo............... .... Peabody & Stearns, Architects.
Church of the Covenant, Washington, D. C................ J. C. Cady & Co., Architects.

COLLEGE AND SEMINARY BUILDINGS.

Harvard Medical School, Boston, Mass .... Ware & Van Brunt, Architects.
Massachusetts Institute of Technology (first building)........ Jonathan Preston, Architect.
Harvard Memorial Hall, Cambridge, Mass................. Ware & Van Brunt, Architects.
Hemenway Gymnasium (Harvard College)................... Peabody & Stearns, Architects.

New Law School (Austin Hall),
Cambridge, Mass..............H. H. Richardson, Architect.
Osborn Hall (Yale), New Haven,
Conn..................Bruce Price, Architect.
Columbia College, New York....C. C. Haight, Architect.
Union Theological Seminary,
New York...................Lord & Potter, Architects.
Girard College, Philadelphia, Pa., T. U. Walter, Architect.

LIBRARIES.

New Public Library, Boston.....McKim, Mead & White, Architects.
Lenox Library, New York.......R. M. Hunt, Architect.

THEATRES AND MUSEUMS, ETC.

Museum of Fine Arts, Boston,
Mass ....Sturgis & Brigham, Architects.
Academy of Fine Arts, Chicago,
Ill ....Burnham & Root, Architects.
The Auditorium Building,
Chicago, Ill ....Adler & Sullivan, Architects.
Casino Theatre, New York......Kimball & Wisedell, Architects.
Metropolitan Opera House, New
York.... ....J. C. Cady & Co.
Academy of Music, Philadelphia,
Pa ....C. Runge, Architect.
Museum of Fine Arts, St. Louis,
Mo .. ..Peabody & Stearns, Architects.

CLUB HOUSES AND LODGE BUILDINGS.

Algonquin Club House, Boston,
Mass..................McKim, Mead & White, Architects.
Art Club Building, Boston, Mass. W. R. Emerson, Architect.
Calumet Club House, Chicago, Ill. Burnham & Root, Architects.
Masonic Temple, Chicago, Ill....Burnham & Root, Architects.
Woman's Temple, Chicago, Ill ..Burnham & Root, Architects.
Denver Club House, Denver, Col. Varian & Stearner, Architects.
Century Club House, New York..McKim, Mead & White, Architects.
Union League Club House, New
York. ....Peabody & Stearns, Architects.

Art Club Building, Philadelphia, Pa........................Frank Miles Day, Architect.
Masonic Temple, Philadelphia, Pa........................J. H. Windrim, Architect.
Masonic Building, Pittsburgh, Pa., Shepley, Rutan & Coolidge, Architects.

OFFICE BUILDINGS.

Ames Building, cor. School and Washington Sts., Boston......Shepley, Rutan & Coolidge, Architects.
Chamber of Commerce, Boston...Shepley, Rutan & Coolidge, Architects.
Fiske Building, Boston, Mass....Peabody & Stearns, Architects.
N. Y. Mutual Life Insurance Company's Building, Boston .....Peabody & Stearns, Architects.
Board of Trade Building, Chicago, Ill........ ....W. W. Boyington, Architect.
Pullman Building, Chicago, Ill..S. S. Beman, Architect.
The Rookery, Chicago, Ill......Burnham & Root, Architects.
Chamber of Commerce, Cincinnati, O......................H. H. Richardson, Architect.
The New York Life Insurance Company's Building, Denver, Col.........................Andrews, Jaques & Rantoul, Architects.
Board of Trade Building, Kansas City.......................Burnham & Root, Architects.
New England Building, Kansas City........................Bradley, Winslow & Wetherell, Architects.
The New York Life Insurance Company's Building, Kansas City........................McKim, Mead & White, Architects.
Equitable Building, New York ..George B. Post, Architect.
N. Y. Mutual Life Insurance Company's Building, New York ...C. W. Clinton, Architect.
Produce Exchange Building, New York......................George B. Post, Architect.
Times Building, New York .....George B. Post, Architect.
United Bank Build'g, New York. Peabody & Stearns, Architects.
World Building, New York.....George B. Post, Architect.

D. O. Mills Block, San Francisco, Cal........................Burnham & Root, **Architects.**

New York Life Insurance Company's Buildings, Montreal, St. Paul, and Minneapolis.......Babb, Cook & **Willard, Architects.**

HOTELS AND APARTMENT HOUSES.

Revere House, Boston...........William Washburn, **Architect.**

Tremont House, Boston.........William Washburn, **Architect.**

The Hollendon Hotel, Cleveland, Ohio........................George F. Hammond, **Architect.**

Midland Hotel, Kansas City, Mo., Burnham & Root, **Architects.**

Aurelia Apartment House, Fifth Avenue, New York..........D. & J. Jardine, **Architects.**

Fifth Avenue Hotel, Fifth Ave., New York ....William Washburn, **Architect.**

The Hotel Imperial, Broadway and 32d St., New York ....McKim, Mead & White, **Architects.**

The Yosemite, Park Ave., New York ....McKim, Mead & White, **Architects.**

Victoria Hotel, New York.......William Washburn, **Architect.**

Hotel Ontario, Salt Lake City ...Adler & Sulivan, **Architects.**

Hotel Ponce de Leon, St. Augustine, Fla ....Carrère & Hastings, **Architects.**

The Alcazar, St. Augustine. Fla., Carrère & Hastings, **Architects.**

RESIDENCES.

House of Ross Winans, Baltimore, Md.......................McKim, Mead & White, **Architects.**

House of F. L. Ames, Boston, Mass ....Sturgis & Brigham, **Architects.**

House of J. F. Andrew, Boston, Mass ....McKim, Mead & White, **Architects.**

House of Cornelius Vanderbilt, New York.....................George B. Post, **Architect.**

House of W. H. Vanderbilt, New York.......................Herter Bros., **Atwood & Snook, Architects.**

House of W. K. Vanderbilt, New York.................R. M. Hunt, Architect.
Houses of Henry Villard, New York........................McKim, Mead & White, Architects.
House of Louis C. Tiffany, New York........................McKim, Mead & White, Architects.

MISCELLANEOUS.

Boston and Providence Railroad Station, Boston ....Peabody & Stearns, Architects.
Fifth Avenue Riding School, New York.......................Carl Pfeiffer, Architect.
Broad Street Railroad Station, Philadelphia, Pa. ....Wilson Bros. & Co., Architects.
Central Railroad Station, Providence, R. I...................T. A. Tefft, Architect.

COST OF BUILDINGS PER CUBIC FOOT.

The most accurate method of estimating the cost of any proposed building, before the plans and specifications are sufficiently complete for taking off the actual quantities, is by means of the cubic contents.

Two buildings built in the same style, and for the same purpose, of the same materials, and on the same scale of wages and prices of materials, should cost the same, or very nearly the same, per cubic foot, although one building be somewhat larger than the other and of different shape.

It therefore follows that if we know the cost per cubic foot of different classes of buildings, in different localities, we can approximate quite closely the cost of any proposed building by multiplying its cubic contents in feet by the known cost per cubic foot of a similar building already built in that locality.

Conversely, if the cost of a proposed building must be kept absolutely within a certain sum, the size of the building should be proportioned so that the cubic contents shall not exceed the quotient obtained by dividing the amount appropriated by the average cost per cubic foot of similar buildings. Even then it may be found, when the bids are opened, that they exceed the appropriation, but the excess will probably not be so great but that the necessary reductions can be made without altering the main features of the building.

In estimating the cost by the cubic contents, it is of course necessary that the contents be computed on the same basis, in both the proposed building and the one already built. In the following examples, the cubic contents are computed from the basement or cellar floor, to the average height of a flat roof, or, if a pitch roof, the finished portion of the attic is included, or that part which might be finished, but mere air-spaces and open porches are not included. Vaults and areas under sidewalks etc., are included as part of the basement. All measurements are to the outside of the walls and foundations. Cost does not, as a rule, include the architect's fee. A few of the examples, that were not compiled by the author, may not be computed closely by the above rule, but it is to be presumed that they are.

The contents of the Government buildings include *all* space, whether finished or not included within the outside lines of the walls and roof, and above the cellar bottom, including all areas and foundations.

The *cost* of the Government buildings does not include the heating apparatus, vaults, site, and approaches.

EXAMPLES OF THE COST OF BUILDINGS PER CUBIC FOOT.

COMPILED BY THE AUTHOR.

NAME OF BUILDING.	DATE.	CHARACTER OF CONSTRUCTION AND FINISH.	COST PER CU. FT.
Chamber of Commerce, Boston, Mass.	1891–2	Seven stories; pitch roof, iron and slate; granite walls, pile foundation; fire-proof construction; marble and oak finish.	29 cts.
"Ames Building," Boston.	1889–91	Thirteen stories; granite and Ohio stone fronts; flat roof; fire-proof construction; marble and oak finish.	53
Exchange Building, Boston.	1889–91	Nine stories; granite front; flat roof; fire-proof construction; marble and oak finish.	40
United States Trust Co. Building, New York.	1888	Ten stories; flat roof; massive granite front; fire-proof construction; extra foundation; fixtures, rich marble work and finish.	60
Seven-story Office Building, New York (R. W. Gibson).	1890	Two massive stone fronts; fire-proof construction; usual machinery, fixtures, etc., complete.	37
Six-story Office Building, New York (R. W. Gibson).		Three brick and terra cotta fronts; non-fire-proof, but with metal lathing; terra-cotta furring; machinery, elevators, etc.	26
Herald Building, New York City.	1893	Two stories and basement; tile and fire-proof roof, brick and stone fronts; fire-proof construction.	46
Auditorium Building, Chicago.	1887–9	(See description, p. 601.)	36
Rookery Building, Chicago.	1886	Eleven stories; flat roof; fire-proof construction; oak finish, marble floor and wainscot; eleven elevators.	32
Masonic Temple, Chicago.	1891	Twenty stories; pitch roof; granite and terra-cotta fronts; skeleton construction; fire-proof; rich marble and metal work; fourteen elevators.	58
Old Colony Building, Chicago.	1893–4	Seventeen stories; flat-roof; Bedford stone, white brick, and terra-cotta fronts; skeleton construction; fire-proof; rich marble and metal work; six elevators.	41
N. Y. Life Insurance Building, La-Salle and Monroe Streets, Chicago.	1893–4	Twelve stories; flat roof; first three stories dressed granite; terra-cotta above; riveted skeleton construction; fire-proof; machinery; rich marble work and finish; small vaults; five elevators.	47
Schiller Building, or German Theatre, Chicago.	1891	Seventeen stories; flat roof, faced with terra-cotta; skeleton construction; fire-proof; rich marble work; theatre in four stories.	$30\frac{8}{10}$

NAME OF BUILDING.	DATE.	CHARACTER OF CONSTRUCTION AND FINISH.	COST PER CU. FT.
Stock Exchange Building, La Salle and Washington Streets, Chicago.	1893–4	Thirteen stories; flat roof; skeleton construction; fire-proof; rich terra-cotta facing.	35½ cts.
Board of Trade Building, Montreal, Can.	1892–3		20
Chamber of Commerce, Cincinnati.	1887–8	Pitch roof; seven stories; granite fronts; fire-proof construction.	26
Wainwright Building, St. Louis.	1890	Ten stories; flat roof; stone facing first and second stories; rich terra-cotta above; skeleton construction; fire-proof; four elevators.	24$\frac{9}{10}$
Union Trust Building, St. Louis.	1892	Fourteen stories; flat roof; rich terra-cotta facing; skeleton construction; fire proof.	
Equitable Building, Denver.	1891–2	Nine stories; flat roof; granite front two stories; light brick and terra-cotta above; fire-proof construction; rich marble work; eight elevators.	42
Ernest and Cramer Building, Denver.	1890	Eight stories; flat roof; brick front; mill construction; oak finish; three elevators.	19
Bailey Block, Denver.	1890	Three stories; flat roof; one front, stone facing; ordinary brick and timber construction; plumbing and steam heat; pine finish.	8½
Crocker Building, San Francisco.	1890	Ten stories; flat roof; brick and terra-cotta fronts; skeleton construction; fire-proof; elaborate finish, marble, etc.	68
Bradbury Building, Los Angeles, Cal.	1891	Five stories; flat roof; buff brick and terra-cotta walls; fire-proof construction; oak finish; two elevators.	32
Endicott Building, St. Paul, Minn.	1887–9	Seven stories; flat roof; pressed brick front; fire proof construction; marble wainscot; five elevators.	29
[illegible] Office Building, Connecticut (R. W. Gibson).	1891	Three stories; two stone fronts; fire proof; usual plumbing, heating plant, fixtures, etc.; rich marble work; stories of moderate height.	50

Hotels and Apartment Buildings.

[illegible] Hotel, New York (R. W. Gibson).		Fourteen stories; brick and terra-cotta front; skeleton construction, riveted; fire proof; usual plumbing, machinery, etc.	44 cts.
Brown Palace Hotel, Denver.	1892	Triangular plan; three stone fronts; considerable carving; nine stories; flat roof; all rooms face street; 350 guest rooms, 160 private baths, 17 public toilet rooms, all tiled; steel construction; fire-proof; provided with electric light, ice and refrigerator plant; laundry; 4 elevators.	30

NAME OF BUILDING.	DATE.	CHARACTER OF CONSTRUCTION AND FINISH.	COST PER CU. FT.
The Lenox (Apartments), Cleveland, O.		Five stories; flat roof; pressed-brick front; partly slow-burning construction.	$18\frac{1}{2}$ cts.

Club Buildings, Y. M. C. A., etc.

Athletic Club Building, Denver, Colo.	1890–1	Four stories; flat roof; one front pressed brick; thoroughly equipped with swimming and Turkish baths, gymnasium, hand-ball room, billiard-room, social rooms, etc.; brick walls, wood construction.	18 cts.
Denver Club Building, Denver, Colo.	1887–8	Three stories and high pitch roof; stone ashlar, four sides; slate roof; wood construction; oak and pine finish.	24
Standard Club Ho., Michigan Avenue, Chicago.	1887		$12\frac{9}{10}$
Y. M. C. A. Building, Cleveland, O.			13

Libraries.

Public Library, New London, Conn.	1889–90	One-story stone building; ordinary construction.	$36\frac{1}{2}$ cts.
Howard Memorial Library, New Orleans, La.	1888		44

Hospitals.

—— Hospital Building, New York (R. W. Gibson).		Seven stories; pressed-brick front; stone trimmings; fire-proof; thorough heating and ventilating plant; plumbing; much marble and tiling.	40 cts.
—— Hospital Building, New York (R. W. Gibson).		Six stories; pressed-brick front; stone trimmings; part fire-proof and part non-fire-proof, but with metal lathing and terra-cotta furring; plumbing, steam plant, etc.	32

Churches.

Grace M. E. Church, Cambridgeport, Mass.	1886–7	Two-story wooden building; tower and spire; slate roof; copper metal work; cost includes furnaces, pews, frescoing, and gas fixtures.	$8\frac{3}{4}$ cts.

NAME OF BUILDING.	DATE.	CHARACTER OF CONSTRUCTION AND FINISH.	COST PER CU. FT.
Christ M. E. Church, Denver, Colo.	1889-91	Two-story stone church; stone tower seventy-one feet high, with wood spire 108 feet high above; shingle roof; steam heat; oak finish in second story; pews, frescoing, etc.	21 cts.
Zion Temple, Synagogue, Ogden Av., Chicago.	1885		7 [illegible]/10

Miscellaneous.

Ursuline Convent, Cleveland, O.	1890	Three stories; pitch roof; brick with stone trimmings; ordinary wood construction.	15 cts.
Hill Theological Seminary, St. Paul, Minn.		Six buildings grouped around a quadrangle; ordinary construction; library, gymnasium, and staircases fire-proof; corridor walls face brick; oak finish; cost per cubic foot *above grade*.	11
Wingate Hall, State College, Orono, Me.	1891-2	Three stories and basement; recitation and drawing rooms; brick with granite trimmings; slate roof; wood floors; brick partitions.	10[illegible]
Grammar School Building, Denver, Colo		Two stories and basement; eight rooms; pressed brick walls; shingle and tin roof; wooden floors; brick partitions; cost, basement floor to second story ceiling.	9[illegible]
Leland Stanford Jr. Museum, Palo Alto, Cal.	1891	This building, covering 21,000 feet and containing over 1,100,000 cubic feet of space, is built entirely of Portland cement concrete—walls, floors, and roof—and is fire-proof throughout.	18

"Fire-proof" denotes iron construction, fire-proofed.

Dwellings.

City dwellings in Chicago, designed by Adler & Sullivan, Architects. Cost per cubic foot from .. .. 17 to 20 cts.

Of dwellings designed by the author and built in Boston in 188[illegible] the average cost of eight and ten room wooden houses per cubic foot of habitable space, including cellar, was about 11

In Denver Colo., the cost of a first-class stone house (isolated), with hard wood finish, indirect steam heat, extra plumbing, decorations, etc., complete, was in 1890 about 27

Brick houses of ten rooms, pine finish, furnace heat, good plumbing, etc., cost above cellar floor, but not including unoccupied roof space, in 1892.................. 14 cts.

Cheap eight-room brick cottages of one and one-half or two stories; bath-room and furnace; cubic space reckoned from cellar floor, but not including unoccupied roof space, can be built in Denver, 1894, for.............. 10

EXHIBIT OF ACTUAL COST AND CONSTRUCTION OF U. S. GOVERNMENT BUILDINGS.

City and Building.	Entire Building. No. of stories.	Entire Building. Cost per cube foot. Cents.	Facing of Walls.
Albany, N. Y. Custom house and Post-office	B. 3 & A.	43.5	Granite, cut ashlar, and moulded courses.
Austin, Tex. Court house and Post-office	B. & 2	42.9	White sandstone, rubbed and moulded.
Chicago, Ill. Custom-house	B. & 4	64.7	Buena Vista sandstone, rubbed and moulded.
Cincinnati, O. Custom-house and Post-office	B. 4 & A.	62.5	Basement, red granite; superstr., gray granite, cut and moulded.
Cleveland, O. Custom-house, etc., extension	B. & 3	45.8	Sandstone, rubbed and moulded.
Evansville, Ind. Custom-house, etc.	B. 2 & A.	35.7	Limestone, moulded trims., rock-face ashlar and granite columns.
Fall River, Mass. Custom house and Post-office	B. & 3	49.2	Gray and red granite, rock-face ashlar and cut trimmings.
Grand Rapids, Mich. Court house and Post-office	B. & 3	24.2	Pressed brick and cut-stone cornices and trimmings.
Hartford, Conn. Custom house and Post-office	B. 3 & A.	100	Granite, cut ashlar and moulded courses.
Kansas City, Mo. Custom-house and Post-office	B. & 3	44.5	Warrensburgh sandstone, rubbed and moulded courses.
Lincoln, Neb. Custom-house and Post office	B. 3 & A.	40.2	Sandstone, rock-face ashlar, rubbed quoins and trimmings.
Little Rock, Ark. Custom-house, Post office, etc.	B. & 3	43.6	Sandstone, rubbed and moulded courses.
Nashville, Tenn. Custom house, etc	B. 3 & A.	35.8	Limestone, rock-face ashlar and rubbed and moulded trimmings.
Parkersburg, W. Va. Custom-house, Post-office, etc.	B. 3 & A.	37.5	Pressed brick, limestone cornices and trimmings.
Philadelphia, Pa. Custom-house and Post-office	B. & 4	58.5	Granite, cut and moulded courses.
Raleigh, N. C. Court house and Post-office	B. 3 & A.	55.8	" " " "
San Francisco, Cal. Appraisers' Stores	B. & 4	34.8	Brick and stone cornices and trimmings.
St. Louis, Mo. Custom house, etc	Sub B. B. & 4	99.8	Basement, red granite; superstr., gray granite, cut and moulded.
Topeka, Kan. Custom house and Post office	B. & 3	37.1	Limestone, rubbed and moulded courses.
Trenton, N. J. Custom house and Post-office	B. & 3	49.5	Amherst sandstone, rubbed and moulded courses.
Washington. Eng. and Printing Bureau	Sub B. B. & 3	15.6	Eng. and Prtg. Bureau. Pressed brick, stone and terra-cotta trims.

COST OF BUILDINGS PER SQUARE FOOT.

One-story buildings of large area, such as exposition buildings, ;c , may be estimated almost as accurately by the square foot as y the cubic foot, as there are no interior partitions, etc.

The cost of the *World's Fair buildings* per square foot of ground ›vered, including sculpture and decoration, as given by E. C. hankland, chief engineer, was as follows :

Manufactures and Liberal Arts Building	$1.39
Transportation Building	1.08
Electricity Building	1.69
Machinery Hall	2.12
Agricultural Building	1.44
Administration Building	9.18
Horticultural Building	1.41
Mines and Mining Building	1.04
Fisheries Building	2.35
Forestry Building	.75

CHARGES AND PROFESSIONAL PRACTICE OF ARCHITECTS.

[As endorsed by the American Institute of Architects at its Annual Convention in 1884.]

GENERAL PROVISIONS.

For full professional services (including supervision), five per cent. upon the cost of the work.

In case of the abandonment of the work, the charge for partial service is as follows:

Preliminary studies.............................. 1 per cent.
Preliminary studies, general drawings, and specifications.............................. 2½ per cent.
Preliminary studies, general drawings, specifications, and details.............................. 3½ per cent.

For works that cost less than $10,000, or for monumental and decorative work, and designs for furniture—a special rate in excess of the above.[1]

For alterations and additions—an additional charge to be made for surveys and measurements

An additional charge to be made for alterations or additions in contracts or plans, which will be valued in proportion to the additional time and services employed.

Necessary travelling expenses to be paid by the client.

Time spent by the architect in visiting for professional consultation and in the accompanying travel, whether by day or night, will be charged for, whether or not any commission, either for office work or supervising work, is given.

The architect's payments are successively due as his work is completed, in the order of the above classifications.

Until an actual estimate is received, the charges are based upon the proposed cost of the works, and the payments are received as instalments of the entire fee, which is based upon the actual cost

The architect bases his professional charge upon the entire cost, to the owner, of the building when completed, including all the fixtures necessary to render it fit for occupation, and is entitled to

[1] Nearly all architects of recognized standing in their profession, charge from 5 to 10 per cent. extra for designing mantels and other ornamental fixtures, carved work, and decorative work of all kinds. Fifteen per cent. on their cost is a common charge for selecting carpets, furnishings, etc.

additional compensation for furniture or other articles designed or purchased by the architect

If any material or work used in the construction of the building be already upon the ground, or come into possession of the owner without expense to him. the value of said material or work is to be added to the sum actually expended upon the building before the architect's commission is computed.

SUPERVISION OF WORKS.

The supervision or superintendence of an architect (as distinguished from the continuous personal superintendence which may be secured by the employment of a clerk-of-the-works) means such inspection by the architect, or his deputy, of a building or other work in process of erection, completion, or alteration, as he finds necessary to ascertain whether it is being executed in conformity with his designs and specifications or directions, and to enable him to decide when the successive instalments or payments provided for in the contract or agreement are due or payable. He is to determine in constructive emergencies, to order necessary changes, and to define the true intent and meaning of the drawings and specifications, and he has authority to stop the progress of the work and order its removal when not in accordance with them.

CLERK-OF-THE-WORKS.

On buildings where it is deemed necessary to employ a clerk-of-the-works, the remuneration of said clerk is to be paid by the owner or owners, in addition to any commissions or fees due the architect.

The selection or dismissal of the clerk of the works is to be subject to the approval of the architect.

EXTRA SERVICES.

Consultation fees for professional advice are to be paid in proportion to the importance of the questions involved, at the discretion of the architect.

None of the charges above enumerated cover professional or legal services connected with negotiations for site, disputed party-walls, right of light, measurement of work, or services incidental to arrangements consequent upon the failure of contractors during the performance of the work. When such services become necessary they shall be charged for according to the time and trouble involved.

DRAWINGS AND SPECIFICATIONS.

Drawings and specifications, as instruments of service, are the property of the architect.

At the Second Annual Convention of the (re-organized) American Institute of Architects, held at Washington, D. C., October 22-25, 1890, the committee on Code of Ethics recommended the adoption of the following clauses to define the superintendence of the architect, and that the Institute adopt the form of contract between owner and architect, given below. The report of the committee was accepted and ordered printed, to be finally considered at the next convention.

SUPERVISION OF WORKS.

"The architect will furnish general superintendence by himself or his deputy, of such frequency or duration as in his judgment will suffice or may be necessary to fully instruct the contractors, pass upon the merits of material and workmanship, and to maintain an effective working organization of the several contractors engaged upon the structure; and to enable him to decide when the successive installments or payments provided for in the contract are due.

"He is to determine any constructive emergencies and order necessary changes, and define the true intent and meaning of the specifications; he has authority to stop the progress of the work and order its removal when not in accordance with them."

"The architect will demand of the contractors the proper correction or remedying of all defects discovered in their work, and will assist the owner in enforcing the terms of the contract. But the architect's superintendence shall not include liability or responsibility for any breach of contract by the contractor."

CLERK-OF-WORKS.

"On buildings where it is deemed necessary to have constant supervision, the architect will, if authorized by the employer, appoint a clerk-of-the works for that purpose, at the extra rates quoted in the schedule or as agreed.

"The selection or dismissal of the clerk-of-the-works is to be subject to the approval of the architect."

"The charge for clerk-of-the-works, when constant supervision is required will be at the rate of $30 per week for buildings costing more than $20,000 and less than $200,000, and at special rates, as agreed, for other buildings."

CONTRACT BETWEEN ARCHITECT AND OWNER.

From ...., Architect,
to ...., Owner.
For a compensation of..,
the architect proposes to furnish preliminary sketches, contract working drawings and specifications, detail drawings and general superintendence of building operations, and, also, to audit all accounts, for a...
to be erected for ....,
on ....

Terms of payment to be as follows :

One-fifth when the preliminary sketches are completed ; three-tenths when the drawings and specifications are ready for letting contracts ; thereafter at the rate of ....per cent. upon each certificate due to the contractor.................................

If work upon the building is postponed or abandoned, the compensation for the work done by the architect is to bear such relation to the compensation for the entire work as determined by the published schedule of fees of the American Institute of Architects.

In all transactions between the owner and contractor, the architect is to act as the owner's agent, and his duties and liabilities in this connection are to be those of agent only.

A representative of the architect will make visits to the building for the purpose of general superintendence, of such frequency and duration as, in the architect's judgment, will suffice, or may be necessary to fully instruct contractors, pass upon the merits of material and workmanship, and maintain an effective working organization of the several contractors engaged upon the structure.

The architect will demand of the contractors proper correction and remedy of all defects discovered in their work, and will assist the owner in enforcing the terms of the contracts ; but the architect's superintendence shall not include liability or responsibility for any breach of contract by the contractors.

The amount of the architect's compensation is to be reckoned upon the total cost of the building, including all stationary fixtures.

Drawings and specifications are instruments of service, and as such are to remain the property of the architect.

..............................., Architect.

Approved and accepted, ..., 189

..................................., Owner.

FORM OF CONTRACT ADOPTED BY THE JOINT COMMITTEE OF THE AMERICAN INSTITUTE OF ARCHITECTS, THE WESTERN ASSOCIATION OF ARCHITECTS, AND THE NATIONAL ASSOCIATION OF BUILDERS.[1]

. . . ., Architect.

THIS AGREEMENT, made the . . . . day of . . . . in the year one thousand . . . . . . hundred and . . . . by and between . . . . part of the first part (hereinafter designated the Contractor), and . . . . part of the second part (hereinafter designated the Owner),

WITNESSETH that the Contractor , being the said part of the first part, in consideration of the covenants and agreements herein contained on the part of the Owner , being the said part of the second part, do covenant, promise and agree with the said Owner , in manner following, that is to say:

1st. The Contractor shall and will well and sufficiently perform and finish, under the direction, and to the satisfaction of . . . . Architect (acting as Agent of said Owner), all the work included in the . . . . agreeably to the drawings and specifications made by the said Architect , and signed by the parties hereto (copies of which have been delivered to the Contractor), and to the dimensions and explanations thereon, therein and herein contained, according to the true intent and meaning of said drawings and specifications, and of these presents, including all labor and materials incident thereto, and shall provide all scaffolding, implements and cartage necessary for the good [illegible] performance of the said work.

2d. Should it appear that the work hereby intended to be done, or any of the matters relative thereto, are not sufficiently detailed or explained on the said drawings, or in the said specifications, the Contractor shall apply to the Architect for such further drawings or explanations as may be necessary, and shall conform to the same as part of this contract, so far as they may be consistent with

[1] Printed here by permission of the Secretary of the Committee, and the Inland [illegible] Co., the licensees for its exclusive publication.

the original drawings, and in event of any doubt or question arising respecting the true meaning of the drawings or specifications, reference shall be made to the Architect , whose decision thereon, being just and impartial, shall be final and conclusive. It is mutually understood and agreed that all drawings, plans and specifications are and remain the property of the Architect .

3d. Should any alterations be required in the work shown or described by the drawings or specifications, a fair and reasonable valuation of the work added or omitted, shall be made by the Architect , and the sum herein agreed to be paid for the work according to the original specification shall be increased or diminished as the case may be. In case such valuation is not agreed to, the Contractor shall proceed with the alteration, upon the written order of the Architect , and the valuation of the work added or omitted shall be referred to three (3) Arbitrators (no one of whom shall have been personally connected with the work to which these presents refer), to be appointed as follows : one by each of the parties to this contract, and the third by the two thus chosen ; the decision of any two of whom shall be final and binding, and each of the parties hereto shall pay one-half of the expense of such reference.

4th. The Contractor shall, within twenty-four hours after receiving written notice from the Architect to that effect, proceed to remove from the grounds or building all materials condemned by , whether worked or unworked, or take down all portions of the work which the Architect shall condemn as unsound or improper, or as in any way failing to conform to the drawings and specifications, and to the conditions of this contract. The Contractor shall cover, protect, and exercise due diligence to secure the work from injury ; and all damage happening to the same by neglect shall be made good by

5th. The Contractor shall permit the Architect , and all persons appointed by the Architect , to visit and inspect the said work, or any part thereof, at all times and places during the progress of the same, and shall provide sufficient, safe and proper facilities for such inspection.

6th. The Contractor shall and will proceed with the said work, and every part and detail thereof, in a prompt and diligent manner, and shall and will wholly finish the said work according to the said drawings and specification, and this contract, on or before the ....day of.......... in the year one thousand.......... hundred and .... (provided that possession of the premises be given the Contractor , and lines and levels of the

building furnished him, on or before the .... day of .... in the year one thousand .... hundred and ....), and in default thereof the Contractor shall pay to the Owner ....dollars for every day thereafter that the said work shall remain unfinished, as and for liquidated damages ....

7th. Should the Contractor be obstructed or delayed in the prosecution or completion of the work by the neglect, delay, or default of any other contractor; or by any alteration which may be required in the said work; or by any damage which may happen thereto by fire, or by the unusual action of the elements, or otherwise; or by the abandonment of the work by the employees through no default of the Contractor, then there shall be an allowance of additional time beyond the date set for the completion of the said work: but no such allowance shall be made unless a claim is presented in writing at the time of such obstruction or delay. The Architect shall award and certify the amount of additional time to be allowed; in which case the Contractor shall be released from the payment of the stipulated damages for the additional time so certified and no more. The Contractor may appeal from such award to arbitrators constituted as provided in Article 3d of this contract.

8th. The Contractor shall not let, assign, or transfer this contract, or any interest therein, without the written consent of the Architect.

9th. The Contractor shall make no claim for additional work unless the same shall be done in pursuance of an order from the Architect, and notice of all claims shall be made to the Architect in writing within ten days of the beginning of such work.

10th. The Owner agree to provide all labor and materials not included in this contract in such manner as not to delay the material progress of the work, and in the event of failure so to do, thereby causing loss to the Contractor, agree that will reimburse the Contractor for such loss: and the Contractor agree that if shall delay the material progress of the work so as to cause any damage for which the Owner shall become liable (as above stated), then shall make good to the Owner any such damage over and above any damage for general delay herein otherwise provided; the amount of such loss or damage, in either case, to be fixed and determined by the Architect or by arbitration, as provided in Article 3d.

11th. The Owner shall effect insurance on said .... work, in own name and in the name of the Contractor , against loss or damage by fire, in such sums as may from time to time be agreed upon with the Contractor , the policies being made to cover work incorporated in the building, and materials for the same in or about the premises, and made payable to the parties hereto, as their interest may appear.

12th. Should the Contractor at any time refuse or neglect to supply a sufficiency of properly skilled workmen, or of materials of the proper quality, or fail in any respect to prosecute the work with promptness and diligence, or fail in the performance of any of the agreements on part herein contained, such refusal, neglect or failure being certified by the Architect , the Owner shall be at liberty, after three days' written notice to the Contractor , to provide any such labor or materials, and to deduct the cost thereof from any money then due or thereafter to become due to the Contractor under this contract; and if the Architect shall certify that such refusal, neglect, or failure is sufficient ground for such action, the Owner shall also be at liberty to terminate the employment of the Contractor for the said work and to enter upon the premises and take possession of all materials thereon, and to employ any other person or persons to finish the work, and to provide the materials therefor; and in case of such discontinuance of the employment of the Contractor he shall not be entitled to any further payment under this contract until the said work shall be wholly finished, at which time, if the unpaid balance of the amount to be paid under this contract shall exceed the expense incurred by the Owner in finishing the work, such excess shall be paid by the Owner to the Contractor , but if such expense shall exceed such unpaid balance, the Contractor shall pay the difference to the Owner . The expense incurred by the Owner as herein provided, either for furnishing materials or for finishing the work and any damage incurred through such default, shall be audited and certified by the Architect , whose certificate thereof shall be conclusive upon the parties.

13th. And it is hereby mutually agreed between the parties hereto that the sum to be paid by the Owner to the Contractor for said work and materials shall be............................

....

subject to additions or deductions on account of alterations as hereinbefore provided, and that such sum shall be paid in current funds by the Owner to the Contractor in installments, as follows:

....

....

It being understood that the final payment shall be made within .... days after this contract is completely finished; provided, that in each of the said cases the Architect shall certify in writing that all the work upon the performance of which the payment is to become due has been done to satisfaction: and provided further, that before each payment, if required, the Contractor shall give the Architect good and sufficient evidence that the premises are free from all liens and claims chargeable to the said Contractor ; and further, that if at any time there shall be any lien or claim for which, if established, the Owner or the said premises might be made liable, and which would be chargeable to the said Contractor , the Owner shall have the right to retain out of any payment then due or thereafter to become due, an amount sufficient to completely indemnify against such lien or claim, until the same shall be effectually satisfied, discharged, or cancelled. And should there prove to be any such claim after all payments are made, the Contractor shall refund to the Owner all moneys that the latter may be compelled to pay in discharging any lien on said premises, made obligatory in consequence of the former's default.

14th. It is further mutually agreed between the parties hereto that no certificate given or payment made under this contract, except the final certificate or final payment, shall be conclusive evidence of the performance of this contract, either wholly or in part, against any claim of the Owner , and no payment shall be construed to be an acceptance of any defective work.

15th. And the said Owner hereby promise and agree with the said Contractor to employ, and hereby employ to provide the materials and to do the said work according to the terms and conditions herein contained and referred to, for the price aforesaid, and hereby contract to pay the same at the time, in the manner, and upon the conditions above set forth.

16th. And the said parties for themselves, their heirs, executors, administrators, and assigns, do hereby agree to the full performance of the covenants herein contained.

....

....

In Witness Whereof, the parties to these presents have hereunto set their hands and seals, the day and year first above written.

In presence of

ARCHITECTURAL SCHOOLS AND CLASSES IN THE UNITED STATES.

The Massachusetts Institute of Technology offers a complete course in architecture extending through four years. No special course is allowed, except in the case of students who may be already qualified for advanced standing in the department; such students must have at least two years' experience in an architect's office, or be over twenty-four years of age, or be graduates of colleges. The course in architecture is intended to furnish both a liberal education and a thorough professional training, aiming not only to prepare its pupils for their years of work as subordinates, but also for their subsequent independent career, when the value of technical knowledge will become most important. Candidates for admission to the first-year class must have attained the age of seventeen years, and must pass satisfactory examinations in arithmetic, including the metric system, algebra, plane geometry, French (German may be offered as a substitute), rhetoric and composition, history and geography. The examinations may be divided between two successive years if preferred.

The tuition fee is $200 per year, payable in advance—$125 on or before October 10, and $75 on or before February 10.

There are several free scholarships in the school. Graduates receive the degree of Bachelor of Science (S. B.).

Columbia College, New York City.—The School of Mines offers a complete four years' course in architecture, very similar to that afforded by the Massachusetts Institute of Technology. Candidates for admission must be eighteen years of age, and must pass a satisfactory examination in physics, chemistry, free-hand drawing, and bookkeeping, besides those studies required by the Institute of Technology.

The examination may be divided between two successive years if preferred. Students who are not candidates for a degree are permitted to pursue such courses as they may be found qualified to enter upon and the faculty may approve.

The chief part of the instruction, both scientific and technical, including architectural history and elementary design, now occupies three years. The fourth year, in which the studies are largely elective, is devoted to advanced work in design, and in construction and practice.

The tuition is $200 a year, payable one-half at the first of each session. Graduates receive the degree of Bachelor of Philosophy

(Ph. B.) There are several free scholarships, and needy and deserving students may have their tuition remitted under certain conditions.

Cornell University, Ithaca, N. Y., offers a complete course in architecture, extending over four years. Graduates receive the degree of Bachelor of Science in Architecture. Tuition $100 per annum, payable in three installments. Candidates for admission must be sixteen years of age, and pass a satisfactory examination in rhetoric and composition, geography, physiology, arithmetic, plane and solid geometry, algebra, American history, and French, German, or Latin.

University of Pennsylvania, Philadelphia.—*Course in Architecture.* The School of Architecture offers a full theoretical, practical, and artistic course of study, as a foundation for a professional career. The course extends through four years. Students not candidates for a degree may pursue a special course, which can be completed in a much shorter time. Graduates in the full course receive the degree of Bachelor of Science. Tuition is $150 a year. Examination for admission about the same as at Cornell, except that French is the only language required besides English.

University of Illinois, Champaign, Ill.—*College of Engineering.* Course in architecture extends through four years. Graduates receive the degree of Bachelor of Science. Tuition free. There is a matriculation fee on entering the college of $10, and a term fee for incidental expenses of $7.50. Students are not admitted under fifteen years of age. Examination same as at Cornell, omitting French and German.

There is also a special builder's course, for those desiring to fit themselves for foremen and builders, covering a single year only.

University of Minnesota, Minneapolis.—This university has very recently (fall, 1891) established a course in architecture.

Pratt Institute, Brooklyn, N. Y.—*Course in Architectural Drawing.* Extends through two years, aiming to give students sufficient training in drawing, design, and the principles of building construction, to fit them for work in an architect's office. There are both day and evening classes. Tuition is $45 a year for the day classes.

The Art Institute of Chicago.—*Classes in Architecture.* Offers technical instruction at moderate cost to the student of architecture, the draughtsman, and the designer. Students may enter the school at any time, paying from the date of entry. There is no prescribed course, but it is not expected that any considerable part of the course can be accomplished in less than two years. The term is continuous from September 29 to June 13, excepting a recess of one week. The tuition is $25 for twelve weeks, or $75 per year.

The Brooklyn Institute.—*Department of Architecture.* Gives instruction in classes to young architects, and draughtsmen employed in the offices of architects. The subjects taught are free-hand drawing, perspective, geometrical drawing, architectural drawing, coloring, designing, and specifications. The instruction is given to any member of the Institute free of cost. Membership in the Institute is $10 the first year, and $5 a year thereafter.

Cooper Union, New York.—The Cooper Union offers a four years' course in architectural drawing, held in the evening. Two different objects are attained in the course: First, the ability to draw ornamental architectural designs, according to conventional rules; and second, skill in the preparation of working drawings from given dimensions. The school year commences October 3, and ends on the 16th day of April. Application for admission may be made in person or by mail at the office of the Cooper Union, beginning June 15, but not before. Each applicant for admission must be at least fifteen years of age. There is no charge for instruction, and drawing materials may be purchased at the school, at a reduction from the usual retail prices.

Academy of Architecture and Building, St. Louis, Mo. H. Mack, Principal.—This is a private school founded by Mr. Mack, in 1885, and designed more particularly to meet the wants of building tradesmen, offering them such instruction as is necessary to attain the highest proficiency in their trade, and to fully understand the plans and details of complicated buildings. There is also a special course for those desiring to fit themselves for positions as draughtsmen in architects' offices.

Tuition for the regular course is $40 per term of twelve weeks, or $100 for the year. There are several special courses which may be commenced at any time, and for which the tuition varies.

The Boston Architectural Club offers instruction, to its members only, by means of illustrated lectures given twice a month, and practical talks once a week. Problems in design are

These fellowships are open to all graduates of the Department of Architecture of Columbia College less than thirty years of age, and are awarded to the successful competitors in a competition held under the direction of the Professor of Architecture, and of an examination in strains and building construction, quantities, etc. Payments are made by the treasurer on the certification of the Professor of Architecture, in four equal installments of $250 each, on the last days of June, September, December, and March succeeding the date of the awards.

The Travelling Scholarship, established by the *American Architect and Building News* ($500 per annum), is open only to subscribers to the imperial or international editions of that journal (without distinction of sex or color) who may dwell in any part of the United States, under the following general conditions :

1. The applicant must be a citizen of the United States.

2. Subscription to the imperial or international edition must be prepaid *in full* direct to the publishers.

3. At the time of filing notice of intention to compete, the applicant must be between the ages of twenty and twenty-five years. It is desirable that notice should be filed with the editors before May 15th.

4. The applicant must have served for at least two years in offices of members of the American Institute of Architects, or of the Western Association of Architects. A graduate's diploma granted by a technical school will be accepted in lieu of one of these years of office-work.

5. Applicants must undergo examination in :
   a. Drawing—free-hand and mechanical.
   b. Architectural Design.
   c. Elements of Construction.
   d. History of Architecture.
   e. English Composition.
   f. One Foreign Language.
   g. Elements of Sanitation, Heating, and Ventilating.

6. Examinations will be held in November.

7. An examination fee of $5 for the benefit of the examiners will be required.

8. The appointee must take his departure within six weeks of receiving notice of appointment. One hundred dollars will be paid to him at the time of taking ocean passage. and the balance of the scholarship in monthly instalments.

9. The course of travel and study while abroad will be outlined by the editors of the *American Architect.*

LIST OF THE BEST TWENTY BOOKS FOR AN ARCHITECT'S LIBRARY.

[Compiled by the editors of the *American Architect and Building News*, in 1886, from forty-nine lists sent in by the subscribers to that journal.]

I. Fergusson, James (33 votes), *The History of Architecture in all Countries*, 2 vols. New York : Dodd, Mead & Co., 1883. $7.50.

II. Gwilt, Joseph (28 votes), *An Encyclopædia of Architecture*, 1 vol. London : Longmans, Green & Co. $17 to $20.

III. *The American Architect and Building News* (28 votes). Boston : Ticknor & Co. $25, $10, $7, $6.

IV. Viollet-le-Duc, E. (19 votes), *Dictionnaire raisonné de l'Architecture Française, du XI. au XVI. Siècle*, 10 vols. Paris : A. Morel et Cie 200 to 250 francs.

V. Smith, Col. (19 votes), *Notes on Building Construction*, 3 vols. London : Rivingtons, 1875. $13.

VI. Trautwine, J. C. (17 votes), *Civil Engineer's Pocket-book*, 1 vol New York : John Wiley & Sons. $5.

VII. Clark, T. M. (16 votes), *Building Superintendence*, 1 vol. Boston : Ticknor & Co. $3.

VIII. Viollet le-Duc, E. (15 votes), *Discourses on Architecture*, 2 vols. Boston : Ticknor & Co. $15.

IX. Jones, Owen (14 votes), *The Grammar of Ornament*, 1 vol. London : Day & Son, 1856. $25.

X. Rosengarten, A. (12 votes), *Hand-book of Architectural Styles*, 1 vol. London : Chatto & Windus. $2.50.

XI. Kidder, F. E. (11 votes), *Architect's and Builder's Pocket-book*, 1 vol New York : John Wiley & Sons, 1892. $4.00.

XII. Webster, Noah (10 votes), *An Unabridged Dictionary of the English Language*. Springfield, Mass. : W. & C. Merriam.

XIII. Ruskin, John (8 votes), *The Stones of Venice*, 3 vols. New York : Merrill & Baker. $4.50.

XIV. Gillmore, J. Q. A. (8 votes), *Practical Treatise on Limes, Hydraulic Cements, and Mortars*, 1 vol. New York : D. Van Nostrand, 1875 $4.

XV. Ware, W. R. (8 votes), *Modern Perspective*, 1 vol., plates in portfolio. Boston : Ticknor & Co. $8.

XVI. Baldwin, W. J. (8 votes), *Steam-Heating for Buildings*, 1 vol New York : John Wiley & Sons. $2.50.

XVII. *The Builder* (7 votes). London : 1843-86. $6.33.

XVIII. Haswell, C. H. (7 votes), *Engineer's and Mechanics' Pocket book* 1 vol. New York : Harper & Bros. $4.

XIX Billings, J. S. (7 votes), *Ventilation and Heating*, 1 vol. New York : *Sanitary Engineer*, 1884.

XX. Ruskin, John (7 votes), *The Seven Lamps of Architecture*, 1 vol New York : Merrill & Baker. $1.00.

XXI. Parker, J. H. (7 votes), *Concise Glossary of Architecture*, 1 vol. Oxford and London : J. Parker & Co. $6.

To the above list we would add the following as being valuable works on the subjects treated :

Lanza, Gaetano, *Applied Mechanics*, cuts. New York (53 East Tenth Street) : J. Wiley & Sons. $7.50.

Thurston, Robert H., *Materials of Construction*. New York (53 East Tenth Street) : J. Wiley & Sons. $5.

Greene, Charles E., *Graphical Analysis of Roof Trusses*. New York (53 East Tenth Street) : J. Wiley & Sons. $1.25.

Birkmire, Wm H., *Architectural Iron and Steel*, cuts. New York (53 East Tenth Street) : J. Wiley & Sons. $3.50.

Baker, Ira O., *Masonry Construction*. New York (53 East Tenth Street) : J. Wiley & Sons. $5.

Newman, John, *Notes on Concrete and Works in Concrete*. New York (12 Cortlandt Street) : E. & F. N. Spon. $1.50.

Blackall, Clarence H , *Builder's Hardware*, cuts. Boston (211 Tremont Street) : Ticknor & Co. $5.

Lloyd, A. Parlett, *Building and Buildings, Building Contracts, etc.* Boston (4 Park Street) : Houghton, Mifflin & Co. $4.50.

Schweinfurth, J. A., *Sketches Abroad*, plates only. Boston (211 Tremont Street) : Ticknor & Co. $15.

Merrill, George P., *Stones for Building and Decoration*, cuts. New York (53 East Tenth Street) : J. Wiley & Sons. $5.

STEAM-HEATING.

HEAT, FUEL, WATER, STEAM, AND AIR.

Heat is measured in two ways: 1st, by the thermometer, as in ordinary practice; and 2d, by the work which it performs.

The *unit of heat* (sometimes called the British thermal unit) is that quantity of heat which will raise the temperature of one pound of water at or near the freezing-point, 1° Fahrenheit.

A French "*calorie*" is the heat required to raise one kilogramme of water 1° Centigrade, and is equal to 3.96832 British thermal units.

The equivalent in force of the unit of heat is the raising of 772 pounds avoirdupois one foot high, and is called the *mechanical equivalent of heat.*

Various kinds of fuel contain a certain number of thermal units per pound: and the method of heating which will convey the largest number of units to the air to be warmed is the most economical. so far as fuel and heating are concerned. But no method has yet been devised which will utilize more than about 85 per cent of the heat units contained in the fuel.

Fuel.[1] — The value of any fuel is measured by the number of heat units which its combustion will generate. The fuels generally used in heating are composed of carbon and hydrogen. and ash, with sometimes small quantities of other substances not materially affecting its value.

"Combustible" is that portion which will burn, the ash or residue varying from 2 to 36 per cent in different fuels.

The following table gives, for the more common combustibles, the air required for complete combustion, the temperature with different proportions of air, the theoretical value, and the highest attainable value under a steam-boiler, assuming that the gases pass off at 320°, the temperature of steam at 75 pounds pressure, and the incoming draft to be at 60°.

[1] From Steam, published by the Babcock & Wilcox Company, New York and Glasgow.

TABLE OF COMBUSTIBLES.

Kind of Combustible.	Air Required. In Pounds per Pound of Combustible.	Temperature of Combustion. With Theoretical Supply of Air.	With 1½ Times the Theoretical Supply of Air.	With Twice the Theoretical Supply of Air.	With Three Times the Theoretical Supply of Air.	Theoretical Value. In Heat Units per Pound of Combustible	In Pounds of Water evaporated from and at 212°, with 1 Pound Combustible.	Highest Attainable Value under Boiler. With Chimney Draft.	With Blast, Theoretical Supply of Air at 60°, Gas 320°.
Hydrogen	36.00	5750	3860	2860	1940	62032	64 20		
Petroleum	15.43	5050	3515	2710	1850	21000	21.74	18.55	19.90
Carbon { Charcoal, Coke, Anthracite Coal, }	12.13	4580	3215	2440	1650	14500	15.00	13.30	14.14
Coal — Cumberland	12.06	4900	3380	26[illegible]0	1730	15370	15.90	14.28	15.06
" Coking Bituminous,	11 73	5140	3520	2680	1810	15837	16.00	14.45	15.19
" Cannel	11.80	4850	3330	2540	1720	15080	15.60	14.01	14.76
" Lignite	9.30	4600	3210	2490	1670	11745	12.15	10.78	11.46
Peat — Kiln-dried	7.68	4470	3140	2420	1660	9660	10.00	8.92	9.42
" { Air-dried, }	5 76	4000	2820	2240	1550	7000	7.25	6.41	6.78
Wood	6.00	4080	2910	2260	1530	7245	7.50	6.64	7.02
" { Air-dried, 20 per cent water, }	4.80	3700	2670	2100	1490	5600	5.80	4.08	4.30

The effective value of all kinds of wood *per pound*, when dry, is substantially the same. The following are the weights and comparative value of different woods by the cord: —

Kind of Wood.	Weight.	Kind of Wood.	Weight.
Hickory, Shell Bark	4469	Southern Pine	3375
Hickory, Red Heart	3705	Virginia Pine	2680
White Oak	3821	Spruce	2325
Red Oak	3254	New Jersey Pine	2137
Beech	3126	Yellow Pine	1904
Hard Maple	2878	White Pine	1868

The following table of American coals has been compiled from various sources: —

AMERICAN COALS.

Coal. State.	Kind of Coal.	Per Cent of Ash.	Theoretical Value. In Heat Units.	Theoretical Value. In Pounds of Water evap.	Coal. State.	Kind of Coal.	Per Cent of Ash.	Theoretical Value. In Heat Units.	Theoretical Value. In Pounds of Water evap.
Penn.,	Anthracite,	3.49	14,199	14.70	Ill.,	Bureau Co.,	5.20	13,025	13.48
"	"	6.13	13,535	14.01	"	Mercer Co.,	5.60	13,123	13.58
"	"	2.90	14,221	14.72	"	Montauk,	5.50	12,659	13.10
"	Cannel,	15.02	13,143	13.69	Ind.,	Block,	2.50	13,588	14.38
"	Connellsville.	6.50	13,368	13.84	"	Caking,	5.66	14,146	14.64
"	Semi bi'nous.	10.77	13,155	13.62	"	Cannel,	6.00	13,097	13.56
"	Stone's Gas,	5.00	14,021	14.51	Md.,	Cumberland,	13.98	12,226	12.65
"	Youghiogheny,	5.69	14,265	14.76	Ark.,	Lignite,	5.00	9,215	9.54
"	Brown.	9.50	12,324	12.75	Col.,	"	9.25	13,562	14.04
Kentucky.	Caking.	2.75	14,391	14.89	"	"	4.50	13,866	14.35
"	Cannel,	2.00	15,198	16.76	Texas,	"	4.50	12,962	13.41
"	"	14.80	13,360	13.84	Wash. Ter.,	"	3.40	11,551	11.96
"	Lignite.	7.00	9,326	9.65	Penn.,	Petroleum,	–	20,746	21.47

"Slack," or the screenings from coal, when properly mixed, — anthracite and bituminous, — and burned by means of a blower on a grate adapted to it, is nearly equal in value of combustible to coal, but its percentage of refuse is greater.

One pound of *pure carbon*, when completely burned, yields 14,500 heat units.

Water and Steam. — The several conditions of water are usually stated as the solid, the liquid, and the gaseous. Two conditions are covered by the last term; and water should be understood as capable of existing in four different conditions, — the solid, the liquid, the vaporous, and the gaseous.

At and below 32° F., water exists in the solid state, as ice; at 39° F., it reaches its maximum density. At the sea-level, water boils, or vaporizes, at 212° F.: the vapor given off being known as steam.

Superheated Steam. — Steam which has a higher temperature than that normal to its pressure is termed "superheated," or "gaseous." Dr. Siemens found, that, when steam at 212° was heated *separate from water*, it increased rapidly in volume, up to 230°, after which it expanded uniformly, as a permanent gas. The use in any steam-boiler of superheating surface exposed to the heated gases of combustion, is highly objectionable, and is of doubtful efficiency. Steam cannot be superheated when in contact with water.

Sensible and Latent Heat of Steam. — The temperature of steam, as shown by the thermometer, is called its sensible

heat, and this varies with every different pressure; but it is found that steam contains more heat than is shown by the thermometer, and this extra heat is called the *latent heat* of steam.

The following table gives the number of British thermal units in a pound of water at different temperatures below the boiling-point. They are reckoned above 32° F.; for, strictly speaking, *water* does not exist below 32°, and ice follows another law. The table also gives the weight per cubic foot at each temperature, calculated by Rankine's formula.

HEAT UNITS IN WATER, BETWEEN 32° AND 212° F., AND WEIGHT OF WATER PER CUBIC FOOT.

Tempera-ture.	Heat Units.	Weight, lbs. per cub. ft.	Tempera-ture.	Heat Units.	Weight, lbs. per cub. ft.	Tempera-ture.	Heat Units.	Weight, lbs. per cub. ft.
32°F.	0.	62.42	123°F.	91.16	61.68	168°F.	136.44	60.81
35	3.	62.42	124	92.17	61.67	169	137.45	60.79
40	8.	62.42	125	93.17	61.65	170	138.45	60.77
45	13.	62.42	126	94.17	61.63	171	139.46	60.75
50	18.	62.41	127	95.18	61.61	172	140.47	60.73
52	20.	62.40	128	96.18	61.60	173	141.48	60.70
54	22.01	62.40	129	97.19	61.58	174	142.49	60.68
56	24.01	62.39	130	98.19	61.56	175	143.50	60.66
58	26.01	62.38	131	99.20	61.54	176	144.51	60.64
60	28.01	62.37	132	100.20	61.52	177	145.52	60 62
62	30.01	62.36	133	101.21	61.51	178	146.52	60.59
64	32.01	62.35	134	102.21	61.49	179	147.53	60.57
66	34.02	62.34	135	103.22	61.47	180	148.54	60.55
68	36.02	62.33	136	104.22	61.45	181	149.55	60.53
70	38.02	62.31	137	105.23	61.43	182	150.56	60.50
72	40.02	62.30	138	106.23	61.41	183	151.57	60.48
74	42.03	62.28	139	107.24	61.39	184	152.58	60.46
76	44.03	62.27	140	108.25	61.37	185	153.59	60.44
78	46.03	62.25	141	109.25	61.36	186	154.60	60.41
80	48.04	62.23	142	110.26	61.34	187	155.61	60.39
82	50.04	62.21	143	111.26	61.32	188	156.62	60.37
84	52.04	62.19	144	112.27	61.30	189	157.63	60.34
86	54.05	62.17	145	113.28	61.28	190	158.64	60.32
88	56.05	62.15	146	114.28	61.26	191	159.65	60.29
90	58.06	62.13	147	115.29	61.24	192	160.67	60.27
92	60.06	62.11	148	116.29	61.22	193	161.68	60.25
94	62.06	62.09	149	117.30	61.20	194	162.69	60.22
96	64.07	62.07	150	118.31	61.18	195	163.70	60.20
98	66.07	62.05	151	119.31	61.16	196	164.71	60.17
100	68.08	62.02	152	120.32	61.14	197	165.72	60.15
102	70.09	62.00	153	121.33	61.12	198	166.73	60.12
104	72.09	61.97	154	122.33	61.10	199	167.74	60.10
106	74.10	61.95	155	123.34	61.08	200	168.75	60.07
108	76.10	61.92	156	124.35	61.06	201	169.77	60.05
110	78.11	61.89	157	125.35	61.04	202	170.78	60.02
112	80.12	61.86	158	126.36	61.02	203	171.79	60.00
114	82.13	61.83	159	127.37	61.00	204	172.80	59.97
115	83.13	61.82	160	128.37	60.98	205	173.81	59.95
116	84.13	61.80	161	129.38	60.96	206	174.83	59.92
117	85.14	61.78	162	130.39	60.94	207	175.84	59.89
118	86.14	61.77	163	131.40	60.92	208	176.85	59.87
119	87.15	61.75	164	132.41	60.90	209	177.86	59.84
120	88.15	61.74	165	133.41	60.87	210	178.87	59.82
121	89.15	61.72	166	134.42	60.85	211	179.89	59.79
122	90.16	61.70	167	135.43	60.83	212	180.90	59.76

For other pressures than those given in the table, it will be practically correct to take the proportion of the difference between the nearest pressures given in the table.

TABLE OF PROPERTIES OF SATURATED STEAM.[1]

Total Pressure per Square Inch.	Temperature in Fahrenheit Degrees.	Total Heat in Heat Units from Water at 32° F.	Latent Heat in Heat Units.	Density or Weight of One Cubic Ft.	Volume of One Pound of Steam.	Relative Volume or Cub. Ft. of Steam from One Cub. Ft. of Water.	Factor of Equivalent Evaporation from Water at 212°.
1	102	1113.05	1042.964	0.0030	330.36	20620	0.965
2	126.266	1120.45	1026.010	0.0058	172.08	10720	0.972
3	141.622	1125.131	1015.254	0.0085	117.52	7326	0.977
4	153.070	1128.625	1007.229	0.0112	89.62	5600	0.981
5	162.330	1131.449	1000.727	0.0137	72.66	4535	0.984
6	170.123	1133.826	995.249	0.0163	61.21	3814	0.986
7	176.910	1135.896	990.471	0.0189	52.94	3300	0.988
8	182.910	1137.726	986.245	0.0214	46.69	2910	0.990
9	188.316	1139.375	982.434	0.0239	41.79	2607	0.992
10	193.240	1140.877	978.958	0.0264	31.84	2360	0.994
15	213.025	1146.912	964.973	0.0387	25.85	1612	1.000
20	227.917	1151.454	954.415	0.0511	19.72	1220.3	1.005
25	240.000	1155.139	945.825	0.0634	15.99	984.8	1.008
30	250.245	1158.263	938.925	0.0755	13.46	826.8	1.012
35	259.176	1160.987	932.152	0.0875	11.65	713.4	1.015
40	267.120	1163.410	926.472	0.0994	10.27	628.2	1.017
45	274.296	1165.600	921.334	0.1111	9.18	561.8	1.017
50	280.854	1167.600	916.631	0.1227	8.31	508.5	1.021
55	286.897	1169.442	912.290	0.1343	7.61	464.7	1.023
60	292.520	1171.158	908.247	0.1457	7.01	428.5	1.025
65	297.777	1172.762	904.462	0.1569	6.49	397.7	1.027
70	302.718	1174.269	900.899	0.1681	6.07	371.2	1.028
75	307.388	1175.692	897.526	0.1792	5.68	348.3	1.030
80	311.812	1177.042	894.330	0.1901	5.35	328.3	1.031
85	316.021	1178.326	891.286	0.2010	5.05	310.5	1.033
90	320.039	1179.551	888.375	0.2118	4.79	294.7	1.034
95	323.884	1180.724	885.588	0.2224	4.55	280.6	1.035
100	327.571	1181.849	883.914	0.2330	4.33	267.9	1.036
105	331.113	1182.929	880.342	0.2434	4.14	265.5	1.037
110	334.523	1183.970	877.865	0.2537	3.97	246.0	1.038
115	337.814	1184.974	875.472	0.2640	3.80	236.3	1.039
120	340.995	1185.944	873.155	0.2742	3.65	227.6	1.040
125	344.074	1186.883	870.911	0.2842	3.51	219.7	1.041
130	347.059	1187.794	868.735	0.2942	3.38	212.3	1.042
140	352.757	1189.535	864.566	0.3138	3.16	199.0	1.044
150	358.161	1191.180	860.621	0.3340	2.96	187.5	1.046
160	363.277	1192.741	856.874	0.3520	2.79	177.3	1.047
170	368.158	1194.228	853.294	0.3709	2.63	168.4	1.049
180	372.822	1195.650	849.869	0.3889	2.49	160.4	1.051
190	377.291	1197.013	846.584	0.4072	2.37	153.4	1.052
200	381.573	1198.319	843.432	0.4249	2.26	147.1	1.053
250	401.072	1203.735	831.222	0.5464	1.83	114	1.059
300	418.225	1208.737	819.610	0.6486	1.54	96	1.064
350	431.956	1212.580	810.690	0.7498	1.33	83	1.068
400	444.919	1217.094	800.198	0.8502	1.18	73	1.073

[1] **Steam, 14th ed.** Babcock & Wilcox Company, New York and Glasgow.

Air. — Air is a mechanical mixture of *oxygen* and *nitrogen*, the proportion for pure air being 77 per cent of nitrogen and 23 per cent of oxygen, by weight. It also contains about $\frac{1}{2000}$ of its volume of carbonic-acid gas and some watery vapor, and is capable of absorbing any other gas or vapor to a certain extent, distributing them through the whole atmosphere by what is called *the law of diffusion of gases*, — a property which gases have of mixing and diluting, which prevents gases of different specific gravities from stratifying for any considerable time. This property is of the utmost importance to air; for, if any noxious or poisonous gas were to remain separated in the atmosphere, any one breathing it would be instantly killed.

Air at 60° F., and with the barometer at 30 inches, is taken as the standard for the comparison of the weight of gases, itself being considered as unity.

At the temperature of 32°, 13½ cubic feet of air weigh a few grains over one pound avoirdupois.

The expansion of air is nearly uniform at all temperatures, expanding about $\frac{1}{490}$ of its bulk at 32°, and for each increase of *one degree* in temperature.

The following table, giving the volume and weight of dry air, tension and weight of vapor, etc., will be found useful for reference. In this table 1000 cubic feet of dry air is taken for a unit, and the co-efficient of expansion is taken at $\frac{1}{490}$, the air being under constant pressure of 30 inches of mercury. Column 5 is taken from Guyot's tables, Regnault's data.

VOLUME AND WEIGHT OF AIR, AND WEIGHT OF V[illegible] TED AIR.

[illegible] in [illegible] — Air is capable of holding, or absorbing, a [illegible] of water, the proportion depending on the temperature of the air.

The warmer it is, the larger quantity it will hold; and as it becomes cool again, it deposits it, or forms clouds or fogs, which condense on any thing colder than the air, leaving the air, upon raising its temperature, capable of taking up more moisture, to be again deposited in dew or rain. It is this property of air which gives it its drying qualities.

An absolutely dry atmosphere is an almost impossibility. Air at 32° contains, when saturated with moisture, $\frac{1}{160}$ of its weight of water; at 59° it contains $\frac{1}{80}$; at 86° it contains $\frac{1}{40}$; its capacity for moisture being doubled by each increase of 27° F.

Air is said to be "saturated" when it has absorbed all the water it will hold at that temperature. The *tension* of vapors is the elastic force or pressure which they exert on the sides of vessels in which they are contained.

Air, to be healthful, should contain about 75 per cent of the moisture required for saturation.

It requires more heat to raise the temperature of a given quantity of moist air one degree than for dry air; but, unless the air is saturated, this difference is not of much practical importance.

Columns 6 and 7 on opposite page give the weight of vapor in 1000 cubic feet of saturated air, and the weight of displaced air, for different temperatures from 0 to 206°.

The numbers in column 6 are obtained by multiplying the corresponding numbers in column 4 by column 5, and the product by 62.355. Column 7 is obtained from column 6, by multiplying value of column 6 by $\frac{8}{5}$.

Specific Heat of Air.—The specific heat of any substance is the quantity of heat required to raise its temperature one degree, compared with the quantity of heat required to raise the temperature of one pound of water at the same temperature one degree. The specific heat of air, as determined by Regnault, is 0.2374. Hence one thermal unit will raise the temperature of one pound of water or $4\frac{1}{5}$ pounds of dry air (equals 51.7 cubic feet at 32° F.) 1° F. As all air contains more or less moisture, which must also be warmed, 50 cubic feet is generally considered as the equivalent of one pound of water in heating.

As one pound of steam at 0 (gauge) pressure condensed to water gives off 965 thermal units, it is therefore equivalent to warming about 48,000 cubic feet of air one degree.

Heating Apparatus.

A steam-heating plant may be divided into three distinct parts: 1st, the boiler, or steam generator; 2d, the radiators; and 3d, the supply and return pipes connecting the two.

In determining the size of a plant required for a given building, the customary practice is, to first determine the amount of radiating surface required to heat the different rooms and halls; then the size of boiler required to furnish sufficient steam for the radiating surface determined upon; and third, the arrangement and size of the piping.

Radiators. — Radiators are generally made of iron, and may be of any shape that will allow of a good circulation of steam through them, and also permit the air to circulate freely about the outside. It is also desirable that the thickness of the metal shall be only sufficient to give sufficient strength.

Twelve or fifteen years ago most radiators were made of wrought-iron piping, but such radiators are now seldom seen except in old buildings. So many improvements have been made since that time in cast-iron radiators that they have largely driven the pipe radiator out of the market.

Classes of Radiators. — Radiators are divided into three classes: those affording, 1st, direct radiation; 2d, indirect radiation; 3d, direct-indirect radiation.

Direct Radiating Surfaces embrace all heaters placed within a room or hall to warm the air *already in the room.*

Indirect Radiating Surfaces embrace heating surfaces placed outside the rooms to be heated, and should only be used in connection with some system of ventilation.

There are two distinct modes of indirect radiation, — one where all the heating surface is placed in a chamber having one side open to the atmosphere; and a fan located on the other side of the room draws the air through the radiating surfaces, and impels it through tubes or ducts to the various rooms in the building. Such a system is only practical where a thorough system of ventilation is provided, and power to propel the fan night and day. The other and more common method is to provide a separate radiator for each room, located at the bottom of vertical flues, leading to the room. The radiators are generally located in the basement, and provided with tin pipes to conduct the hot air to the rooms. Where the rooms are very large, it will generally be found best to divide the heating surface into two stacks, with separate pipes and registers.

Direct-Indirect radiation is a mean between the other two methods. The radiators are placed in the rooms to be heated, as

in the first method, and a supply of fresh air brought to them through openings in the outside wall of the room, or through a space under the lower sash of a window.

Efficiency of Radiators.—The condensation of one pound of steam at the pressure of one atmosphere to water at 212°, gives out 965 thermal units. Hence, to determine the amount of heat given off by any radiator in a given time, it is only necessary to determine the amount of water in pounds which the radiator condenses in the same time, and multiply it by 965.

The radiator which, under the same conditions of steam-pressure, and volume and temperature of surrounding air, will condense the most water in a given time, is the most efficient.

Heating by Direct Radiation.—Direct radiation being much more economical than indirect radiation, it will always be much more commonly used for steam or hot-water heating; and in buildings not requiring a great amount of ventilation it offers a nearly perfect mode of heating.

Measurement of Radiators.—Radiators are rated, or measured, not according to their size, but according to the amount of heating surface coming in contact with the air. The size of radiator for a given amount of heating surface depends entirely upon the form or shape of the radiator.

The cheapest direct radiator is one formed of wrought-iron pipes, 1-inch pipes being generally preferred, placed against a wall, one above the other, and connected with return bends [illegible] length of pipe required to make up a given amount of heating surface [illegible] easily be determined [illegible] the use of the [illegible] 764. For rooms [illegible] desirable that [illegible] apparatus shall [illegible] appearance, and occupy as little space as possible, [illegible] of upright radiator is generally employed. Fig. 1

Fig. 1.—[illegible] Radiator.

shows a style of radiator, known as a pipe radiator, which was formerly largely used on account of its cheapness; it is now seldom seen, however. Pipe radiators are formed of a number of short, upright, 1-inch tubes, from 2 feet 8 inches to 2 feet 10 inches long, screwed into a hollow cast-iron base or box, and are either connected together in pairs by return bends at their upper ends, or else each tube stands singly, with its upper end closed, and having a hoop-iron partition extending up inside it, from the bottom to nearly the top. The radiators are also made circular in form, either in one piece, or in halves for encircling iron columns.

The following table shows the dimensions of 1-inch pipe radiators for different heating surfaces: —

TABLE OF VERTICAL PIPE RADIATORS.

No. of Rows and Width of Base.	Tubes in Each Row.	Surface, in Sq. Ft.[1]	Length. Ft.	Length. In.	[illegible] and of	Tubes in Each Row.	Surface, in Sq. Ft.[1]	Length. Ft.	Length. In.
Single Rows, Width of Base, 4½ ins.	4	4	0			8	16	1	6½
	6	6	1			10	20	1	10½
	8	8	1			12	24	2	2½
	10	10	1			14	28	2	6½
	12	12	2			16	32	2	10½
	16	16	2			18	36	3	2½
	20	20	3			20	40	3	6½
	24	24	4			24	48	4	2½
	28	28	4			28	56	4	10½
	32	32	5			32	64	5	6½
	38	38	6			38	76	6	6½
Three Rows, Width of Base, 8½ ins.	8	24	1		Width of Base, 10 ins.	4	16	0	10½
	12	36	2			8	32	1	6½
	16	48	2			12	48	2	2½
	20	60	3			16	64	2	10½
	24	72	4			20	80	3	6½
	28	84	4			24	96	4	2½
	32	96	5			28	112	4	10½
	38	114	6			32	128	5	6½

[1] For radiators 35 inches high.

in the first method, and a supply of fresh air brought to them through openings in the outside wall of the room, or through a space under the lower sash of a window.

Efficiency of Radiators.—The condensation of one pound of steam at 0, or pressure of one atmosphere to water at 212°, gives out 965 thermal units. Hence, to determine the amount of heat given out by any radiator in a given time, it is only necessary to determine the amount of water in pounds which the radiator condenses in the same time, and multiply it by 965.

The radiator which, under the same conditions of steam-pressure, and volume and temperature of surrounding air, will condense the most water in a given time, is the most efficient.

Heating by Direct Radiation.—Direct radiation being much more economical than indirect radiation, it will always be much more commonly used for steam or hot-water heating; and in buildings not requiring a great amount of ventilation it offers a nearly perfect mode of heating.

Measurement of Radiators.—Radiators are rated, or measured, not according to their size, but according to the amount of heating surface coming in contact with the air. The size of radiator for a given amount of heating surface will depend entirely upon the form or shape of the radiator.

The cheapest direct radiator is one formed of wrought iron pipes (1-inch pipes being generally preferred) placed against a wall, one above the other, and connected with return bends [illegible] a circulation. [illegible] length of pipe required to make up a given amount of heating [illegible] be deter[illegible] by the use of the [illegible]. For rooms [illegible] desirable that [illegible] apparatus shall [illegible] appearance, and occupy as little space as possible, [illegible] upright radiator is generally employed. Fig. 1

FIG. 1.—DIRECT-PIPE RADIATOR.

shows a style of radiator, known as a pipe radiator, which was formerly largely used on account of its cheapness; it is now seldom seen, however. Pipe radiators are formed of a number of short, upright, 1-inch tubes, from 2 feet 8 inches to 2 feet 10 inches long, screwed into a hollow cast-iron base or box, and are either connected together in pairs by return bends at their upper ends, or else each tube stands singly, with its upper end closed, and having a hoop-iron partition extending up inside it, from the bottom to nearly the top. The radiators are also made circular in form, either in one piece, or in halves for encircling iron columns.

The following table shows the dimensions of 1-inch pipe radiators for different heating surfaces:—

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No. of Rows and Width of Base.	Tubes in Each Row.	Surface, in Sq. Ft.[1]	Length Ft.	Length In.
Single Rows, Width of Base, 4½ ins.	8	16	1	6½
	10	20	1	10½
	12	24	2	2½
	14	28	2	6½
	16	32	2	10½
	18	36	3	2½
	20	40	3	6½
	24	48	4	[illegible]
	28	56	4	10½
	32	64	5	6½
	38	76	6	6½
Three Rows, Width of Base, 8½ ins.	4	16	0	10½
	8	32	1	6½
	12	49	2	2½
	16	64	2	10½
	20	80	3	6½
	24	96	4	2½
	28	112	4	10½
	32	128	5	6½
Four R[illegible] Width o[illegible] 10 [illegible]	24	72	4	2½
	28	84	4	10½
	32	96	5	6½
	38	114	6	6½

[1] For radiators 35 inches high.

The American Company also makes corner radiators, circular, curved, and column radiators, window radiators (height as low as 13 inches), and dining-room radiators (with hot closet) for steam or water, and stairway radiators for steam only. They also make adjustable legs that can be fitted to any of their single loop radiators. Fig. 3 illustrates a curved radiator.

LIST OF SIZES

NATIONAL, IDEAL, PEERLESS, AND PERFECTION STEAM AND WATER RADIATORS.

(*Made by the American Radiator Company*)

No. of Sections.	Length, inches.
2	5
3	7½
4	10
5	12½
6	15
7	17½
8	20
9	22½
10	25
11	27½
12	30
13	32½
14	35
15	37½
16	40
17	42½
18	45
19	47½
20	50
21	52½
22	55
23	57½
24	60
25	62½
26	65
27	67½
28	70
29	72½
30	75
31	77½

Each section of these radiators is 7¾ inches wide. Width of legs 8¼ inches.

Radiators will be tapped 2 inches and bushed unless otherwis ordered.

In estimating length of radiator allow ½ inch for each bushing.

Fig. 3. Perfection Special Wide-top Curved Radiator.

Fig. 4 shows an end view of the new standard radiators made by the Standard Radiator Company. The four-column radiator is 12 inches wide, the three-column 9 inches wide, and the two-column radiator $5\frac{1}{2}$ inches wide. Each section makes $2\frac{1}{2}$ inches in the length of a radiator, *i.e.*, a radiator of ten sections would be 25 inches long; one of sixteen sections, 40 inches long, etc.

The following table gives the heating surface *per section* for the different heights made:

HEATING SURFACE PER SECTION OF NEW STANDARD RADIATORS.

Height in inches:	44	38	32	26	22	16
	SQ. FT.	SQ. FT.	SQ. FT.	SQ. FT.	SQ. FT.	SQ. FT.
2 columns	$3\frac{1}{3}$	3	$2\frac{1}{2}$	2	$1\frac{2}{3}$	$1\frac{1}{4}$
3 columns	6	5	$4\frac{1}{2}$	$3\frac{3}{4}$	3	$2\frac{1}{4}$
4 columns	$8\frac{1}{4}$	7	$5\frac{3}{4}$	$4\frac{1}{2}$	7	$2\frac{3}{4}$

From the above data, the size of a radiator for any required heating surface may be easily computed.

These radiators are made for either steam or hot water.

Union Radiators.—The following table gives the size and radiating surface of the Union and Royal Union radiators, manufactured by the H. B Smith Company. Fig. 5 illustrates the appearance of the Royal Union radiator.

FIG. 5.—ROYAL UNION RADIATOR, FOR STEAM OR WATER.

DIMENSIONS OF UNION AND ROYAL UNION RADIATORS.

Height of Radiators:				37 in.	29 in.	25 in.	21 in.	17 in.
No. of Sections.	Total Width.	Extreme Length.		Radiating Surface.	Radiating Surface.	Radiating Surface.	Radiating Surface.	Radiating Surface.
	In.	Ft.	In.	Sq. Ft.	Sq. Ft.	Sq. Ft.	Sq. Ft.	Sq. Ft.
3	9¾	0	9	13½	10½	9	7½	6
4	9¾	1	0	18	14	12	10	8
5	9¾	1	3	22½	17½	15	12½	10
6	9¾	1	6	27	21	18	15	12
7	9¾	1	9	31½	24½	21	17½	14
8	9¾	2	0	36	28	24	20	16
9	9¾	2	3	40½	31½	27	22½	18
10	9¾	2	6	45	35	30	25	20
11	9¾	2	9	49½	38½	33	27½	22
12	9¾	3	0	54	42	36	30	24
13	9¾	3	3	58½	45½	39	32½	26
14	9¾	3	6	63	49	42	35	28
15	9¾	3	9	67½	52½	45	37½	30
16	9¾	4	0	72	56	48	40	32
17	9¾	4	3	76½	59½	51	42½	34
18	9¾	4	6	81	63	54	45	36
19	9¾	4	9	85½	66½	57	47½	38
20	9¾	5	0	90	70	60	50	40
21	9¾	5	3	94½	73½	63	52½	42
22	9¾	5	6	99	77	66	55	44
23	9¾	5	9	103½	80½	69	57½	46
24	9¾	6	0	108	84	72	60	48
25	9¾	6	3	112½	87½	75	62½	50
26	9¾	6	6	117	91	78	65	52
27	9¾	6	9	121½	94½	81	67½	54
28	9¾	7	0	126	98	84	70	56
29	9¾	7	3	130½	101½	87	72½	58
30	9¾	7	6	135	105	90	75	60

Rules for determining Direct Radiating Surface required for heating various classes of rooms and buildings. The common practice of determining the direct radiating surface required in heating, is to allow one square foot of radiating surface to a certain number of cubic feet to be warmed.

The following proportions may be considered as an average of those recommended by different engineers and experts: —

For dwellings, cold or exposed rooms, 1 foot heating surface to 50 cubic feet; for dwellings, ordinary rooms, 1 foot heating surface to 60 or 70 cubic feet; for dwellings, warm, sunny rooms, 1 foot heating surface to 75 cubic feet; for stores, wholesale, 1 foot heating surface to 125 cubic feet; for stores, retail, 1 foot heating surface to 100 cubic feet; for offices, 1 foot heating surface to 75 cubic feet; for churches and audience-rooms, 1 foot heating surface to 125 to 150 cubic feet; for factories and workshops, 1 foot heating surface to 200 cubic feet.

City houses require less heat than country houses, and brick houses less than wood.

Upper rooms require less heat than those on the ground floor.

Mr. William J. Baldwin, in his excellent work on "Steam-Heating for Buildings,"[1] recommends the following rule, which he has used for several years, and which is not wholly empirical: —

"Divide the difference in temperature between that at which the room is to be kept, and the coldest outside atmosphere, by the difference between the temperature of the steam-pipes and that at which you wish to keep the room; and the product will be the square feet, or fraction thereof, of plate or pipe surface to each square foot of glass, or its equivalent in wall-surface."

The equivalent glass surface is found by multiplying the superficial area of the walls in square feet by the number opposite the substance in the following table, and dividing by 1,000 (the value of glass). The result is the equivalent of so many square feet of glass in cooling power, and should be added to the window surface.

TABLE OF POWER OF TRANSMITTING HEAT OF VARIOUS BUILDING SUBSTANCES, COMPARED WITH EACH OTHER.

Window Glass	1,000
Oak and Walnut	66
White Pine	80
Pitch Pine	100
Lath and Plaster	75 to 100

[1] Published by John Wiley & Sons of New York.

Common Brick (rough)	200 to 250
Common Brick (whitewashed)	200
Granite or Slate	250
Sheet-Iron	1,030 to 1,110

It must be distinctly understood that the extent of heating surface found in this way offsets only the windows and other cooling surfaces *it is figured against*, and does not provide for cold air admitted around loose windows, or between the boarding of poorly constructed wooden houses. These latter conditions, when they exist, must be provided for by additional heating surface.

Example. — What amount of heating surface should be supplied to the sitting-room of a wooden dwelling with two outside walls, one 14 feet by 9 feet high, and the other 15 feet by 9 feet; the total window area being 54 square feet, the external temperature frequently being at 0 F., and the steam never exceeding 5 pounds pressure?

Ans.[1] — Temperature of room, $70° - 0° = 70°$; temperature of steam-pipes at 5 pounds, $228 - 70° = 158$; $70 \div 158 = .443$, or a little less than one-half a square foot of heating surface to each square foot of glass or its equivalent.

Area of outside walls $= 14 \times 9 = 126 + 15 \times 9 = 126 + 135 = 261$. Subtracting the glass area, 54, we have 207 square feet of lath and plaster.

$$\begin{array}{r} 207 \times \quad 100 = 20{,}700 \\ 54 \times 1{,}000 = 54{,}000 \\ \hline 1{,}000)74{,}700 \\ \hline \end{array}$$

Equivalent glass area = 74. Multiplying this by .443, we have 33 as the number of square feet of radiating surface required to warm the room, or 1 foot of surface to 58 cubic feet of air-space.

In practical work, it is well to determine the heating surface by both of the methods given, and then use the larger quantity. There can never be any bad results from having an excess of heating surface, while a deficiency will always result in cold rooms in extreme cold weather.

Direct-Indirect Radiation.

The only difference between this method of heating and the direct method is, that external air is introduced into the room in such a way that it shall come in contact with the radiator, and,

[1] It should be noticed that this proportion *does not depend* upon the size of the room, but only upon the climate, pressure of the steam, and desired temperature of the room.

becoming heated, circulate through the room; and, unless other means are provided, pass out through the cracks around the doors and windows There are several methods of arranging the radiators and cold-air inlets, although nearly all require that the radiator shall be located against an outside wall.

The simplest method of providing direct-indirect radiation is by using a radiator that has the lower portion encased so as to

FIG. 6.—PERFECTION DIRECT-INDIRECT RADIATOR.

form a box, as shown in Fig. 6. Cold air can be conducted from the outside of the house through a galvanized iron pipe, and admitted to the bottom of the radiator. It is then obliged to pass upward between the radiator flues, their entire length, and is brought into the room at an exceptionally high temperature. A small damper door is placed in each end of the box, and a damper should also be put in the cold air supply, so that the radiator can be converted into the ordinary direct type by simply closing the damper and opening the doors. This would probably be required in very cold weather. The outside of the radiator, of course, heats by direct radiation at all times. If a large amount of ventilation is required, some form of indirect radiator should be enclosed in an incombustible casing and the outside air admitted

below the radiator. A very good arrangement to accomplish this purpose is shown in Fig. 7.

It consists of a stack of pin or other indirect radiators, encased in a box of either iron marble, or wood lined with tin, and provided with registers at the top for the escape of the heated air. The cold air enters through a hollow iron sill placed above the wooden sill of a window, down back of the radiator, through a galvanized iron pipe, to the space under the radiator.

The cold-air inlet is provided with a damper, so that it can be closed; and registers are also placed at the base of the radiator casing, so that, in very cold weather, the cold-air inlet may be partially or wholly closed, and the air allowed to circulate through the bottom register, up through the radiator, and out of the top registers.

Longitudinal Elevation of Coil.
FIG. 7.

Indirect Radiation.

Heating by indirect radiation is, as has been previously stated, accomplished by two methods; the more general method being to have separate radiators for each room, located in the cellar or basement, incased with metal or wood lined with tin, and provided with a fresh-air inlet, and tin pipe to convey the hot air to the room to be heated

The other method is, to provide one cold-air inlet for the whole building, and place a large coil of steam-pipes behind it, so that all the air entering the building must pass through this coil. Such a method can only be used in connection with fan-ventilation.

Fig. 8 shows the usual method of casing indirect radiators. The casing is generally of wood lined with tin, or of sheet-metal. The former is best when the cellar is to be kept cool, as there is a greater loss by radiation and conduction through metal cases; otherwise metal is best, as it will not crack, and, when put together with small bolts, can be removed to make repairs, without damage.

The boxes should be fitted with a door on one of the sides, and the cold-air pipe should always be provided with a damper.

The vertical air-ducts are usually tin flues built into the wall when the building is going up. Sometimes they are only plastered; but round, smooth metal linings with close joints give much the best results. The cross-section of an air-duct should be comparatively large, as a large volume of warmed air, with a slow velocity, gives the best result.

There should be a separate vertical air-duct for every outlet or register. In branched vertical air-ducts one is generally a failure.

FIG. 8.—CASING FOR INDIRECT RADIATORS.

The heated air from one heater may be taken to two or more vertical air-ducts, when they start directly over it; but one should not be taken from the top and the other from the side, or the latter will be a total failure, unless the room to which the flue runs is exhausted; i.e., the cold or vitiated air of the room is drawn out by a heated flue or otherwise.

Inlet or cold-air ducts are best when there is one for every coil or heater. Sometimes only one large-branched cold-air duct is

Westfield, Mass. This is a cast-iron radiator, which is very extensively used throughout the country. As there is now no patent on this radiator, and it is comparatively cheap, it is manufactured by many different companies.

The radiator as made by the H. B. Smith Company is made in sections of 10 square feet of heating surface to a section. Each section is 6¼ inches high, 41 inches long, and 8 inches wide, and contains 936 pins, each pin having a base of ½ inch, a top of ¼ inch, and a length of 1 1/16 inch; the pins being in staggered rows, as shown in Fig. 10.

To find the floor-space for any number of sections, allow 8 inches for the width of each section, plus ½ inch for each outside section, and the thickness of the box twice. The more modern styles of pin indirect radiators have the connections at the ends.

Fig 11 shows a stack of five sections of a pin radiator made by the American Company.

The sections made by this company are of two sizes; viz, stand-

FIG 11 PERFECTION PIN INDIRECT RADIATOR.

ard size, 7½ inches wide, 36 inches long, and occupying 2¾ inches in stack, the heating surface being 10 square feet; and extra large, which is 11¼ inches wide, 36 inches long, and occupying 2¾ inches in stack, the heating surface being 15 square feet.

The Standard Radiator Company also makes an improved indirect pin radiator with 12 and 15 feet of heating surface in the sections.

Fig. 12 shows one section of the Excelsior indirect steam radiator made by the American Radiator Company

This radiator has two nearly horizontal pipes or tubes inclined in opposite directions, and connected at the ends so as to form a complete pipe circuit. In one of the ends or upright sections a diaphragm or partition is so arranged as to stop the flow of steam from the inlet directly to the outlet opening, but at the same time allows the water of condensation to pass under it and directly through the radiator, and from radiator to radiator when connected to-

Nearly all indirect radiators can be used either for steam or hot water; and for this reason it is often advantageous to heat dwellings, etc., entirely by indirect radiation, in which case the apparatus may be used for heating by hot water in moderate weather: and, by drawing off the water in cold weather, the pipes and radiators may be filled with steam. This method is now largely employed in first-class city houses.

Rules for computing Indirect Heating Surfaces.

It is quite a common custom among steam-fitters to double the direct radiating surface for indirect radiation, but this is an exceedingly loose method.

In warming by indirect radiation, a fresh supply of air is constantly passing over the radiator, and no air is heated twice. The heated air usually enters the room at from 110° to 130° in hot weather, and, coming in contact with walls, windows, furniture, etc., is quickly cooled to the desired temperature.

It is therefore evident, that, if we can determine the amount of air to be warmed, and, by experiments, the quantity of air that one square foot of indirect radiator will heat under certain conditions, we can easily determine the radiating surface required.

By careful study of the records of various experiments made on indirect heating, and by certain fundamental principles in steam-heating, the author has computed the table following, showing the quantity of air which one foot of indirect radiating surface will warm in an hour, at various steam-pressures, and from 0 and 10° F.

Divide the quantity of air to be heated per hour by the corresponding number in the table, and the result will be the amount of indirect radiating surface required in well-built brick buildings, and in which the window surface is not more than $\frac{1}{50}$ the cubic contents of the room. Where the window surface exceeds this proportion, increase the radiating surface from 10 to 20 per cent. For wooden buildings also, add 10 per cent. The numbers in the columns under "Forced Draught" should not be used unless the air in the room to be heated is changed at least six times an hour; and the quantity of air should never be taken at less than four times the cubic contents of the room.

If the external temperature is liable to be at 0° for any length of time, the fourth and fifth columns should be used. The second and third columns are intended for comparatively warm climates.

QUANTITY OF AIR WARMED PER HOUR BY ONE SQUARE FOOT OF INDIRECT HEATING SURFACE, WITH NATURAL OR FORCED DRAUGHTS.

Steam Pressure above Atmosphere.	Cubic Feet of Air Warmed per Hour.			
	10° to 100° F.		0° to 120° F.	
	Natural Draught.	Forced Draught.	Natural Draught.	**Forced Draught.**
Lbs.	Pipe and Pin.	Pin.	Pipe and Pin.	**Pin.**
0	150	251	125	**208**
3	160	267	133	**223**
5	165	276	138	**229**
10	177	296	148	**246**
20	198	330	165	**275**
30	212	353	177	**294**
60	245	408	204	**340**

Example II. — As an example of indirect heating, we will take the same room as in Example I.: viz., room $15' \times 14' \times 9'$, with 54 square feet of window area; steam-pressure, 5 pounds; location, Massachusetts; wooden house.

Ans. — Cubic contents $= 15 \times 14 \times 9 = 1{,}890$. Multiplying this by 4, we have 7,560 cubic feet of air to be heated per hour. Dividing by 138, taken from column 4, we have 54 as the number of square feet of heating surface required to heat this amount of air. As the building is of wood, and the glass area exceeds $\frac{1}{50}$ of the cubic space, we had better increase the heating surface 10 per cent, making it 60 square feet.

Example III. — What should be the indirect heating surface in a schoolroom $21 \times 32 \times 12$ feet, where the air is changed six times an hour; brick building, situated in Northern States; steam pressure, 5 pounds.

Ans. — $24 \times 32 \times 12 = 9{,}216$. Multiplying by 6, we have 55,296. Dividing this by 229, we have 242 square feet as the required heating surface.

If the room had only natural ventilation, we would multiply the contents by 4, and divide by 138; and we have 260 square feet. Radiators are always more effective, the greater the quantity of air passing over them.

Indirect Radiation, with Plenum Ventilation. — The plenum system of ventilation is produced by forcing warm, fresh air into all the rooms, and by causing a pressure slightly in excess of that of the external atmosphere, forcing the impure air from the room.

This system requires that the whole air-supply of the building shall enter at one point, where it must pass through a large steam radiator, generally a stack of one-inch pipes, and from thence into one large duct, with branches to the various rooms, or into a plenum chamber in the cellar, from which it passes upward, through ducts provided for the purpose, into the rooms above. If the heated air passes directly into a main air-shaft, with branches to the various rooms, it must be heated to required degree before entering the duct, by the single large radiator referred to; but if the air passes into a plenum chamber, it is generally only heated to about 60° by the main radiator, and smaller indirect radiators are located at the foot of the ducts leading to the rooms, to give the air entering the rooms the desired temperature. The latter is much the better way for large buildings, especially theatres, concert halls, churches, etc.

In either case a fan will be required, which must be located just behind the large steam radiator, to draw the air through it, and produce the plenum.

Steam-Boilers.

The capacity of steam-boilers for generating steam is generally designated by the number of *horse-power* of the boiler.

Strictly speaking, there is no such thing as "horse-power" to a steam-boiler, as it is a measure applicable only to dynamic effect. But, as boilers are necessary to drive steam-engines, the same measure applied to steam-engines has come to be universally applied to the boiler, and cannot well be discarded.

At the present time a horse-power is generally measured by the evaporation of 30 pounds of water per hour, at 70 pounds pressure, from feed-water at 100°.

For heating purposes it is more convenient to designate boilers by the square feet of heating surface which they contain. One square foot of heating surface in one form of boiler may, however, be much more efficient than in another style; and the value of a foot of heating surface must be determined by experiment. The following table gives an approximate list of square feet of heating surface per horse-power in different styles of boilers; the rate of combustion of coal per hour, per square foot of fire surface, required for that rating; the relative economy, and the rapidity of steaming:—

Type of Boiler.	Sq. Ft. for One H.P.	Coal for each Sq. Ft.	Relative Economy.	Relative Rapidity of Steaming.	Authority.
Water-tube	10 to 12	0.3	1.00	1.00	Isherwood.
Tubular	14 to 18	0.25	0.91	0.50	"
Flue	8 to 12	0.4	0.79	0.25	Prof. Trowbridge.
Plain Cylinder . . .	6 to 10	0.5	0.69	0.20	"
Locomotive	12 to 16	0.275	0.85	0.55	
Vertical Tubular . .	15 to 20	0.25	0.80	0.60	

In tubular boilers, 15 square feet of heating surface is generally taken as a horse-power.

A horse-power in a steam-engine, or other prime mover, is 550 pounds raised 1 foot per second, or 33,000 pounds 1 foot per minute.

For determining the capacity of a boiler for supplying a given amount of radiating surface, allow one square foot of boiler surface to from 7 to 10 square feet of radiating surface: the proportion depending upon the nature of the radiating surface and the efficiency and size of the boiler.

Small boilers for house-use should be much larger proportionately than large plants. In average buildings in the Northern States, where the building is entirely heated by direct radiation, one square foot of surface in a horizontal tubular boiler, well set, and with the supply and return pipes properly run, will supply 8 square feet of radiating surface. If all indirect radiation is used, this number should be reduced to 6.

Classes of Boilers.—There are a great many kinds of boilers manufactured for heating purposes, and especially for heating dwelling-houses. For dwellings, it is desirable that the boiler shall be safe, provided with automatic dampers, safety-valves, etc., and shall be as simple as possible, and designed to utilize the largest possible percentage of the heat generated by combustion.

For heating large buildings, either a tubular or sectional boiler is generally employed. The former is so common as hardly to need description. It consists of a wrought-iron cylinder with closed ends, with the lower half filled with wrought-iron tubes, which pass through the ends, and are welded to it. When set and ready for use, the boiler is filled to a point a little above the highest row of tubes: the boiler is set so that the products of combustion shall pass under the boiler, and back again through the tubes to the front of the boiler, from whence they pass to the chimney.

Hence the heating surface in a horizontal tubular boiler consists of one-half the area of the shell and ends, and the total external area of the tubes.

The heating surfaces for the various standard sizes manufactured by Kendall & Roberts, of Cambridge, Mass., are given in the table on pp. 808, 809. These surfaces would also apply to boilers of the same dimensions and number of tubes of any other manufacture.

Upright tubular boilers are *filled* with tubes in the same way.

Sectional Boilers are generally made of cast-iron, each section being a boiler by itself. The steam is collected in a common wrought-iron drum, and returned to another drum. The advantage of these boilers is, that no serious explosion can result from them; as, should an explosion occur, it would probably be confined to not more than two sections, which in most boilers can be easily replaced.

These boilers are especially adapted to schools, churches, etc.

Supply and Return Pipes. — The main supply-pipe should be not less than 4 feet above the water-line of the boiler in medium-sized buildings; and in buildings covering a larger area, the height should be as much more than this as it is practical to make it.

Where the condensed water is returned to the boiler, or where low pressure of steam is used, the *diameter of the main* in inches should be equal to *one-tenth of the square root* of the radiating surface supplied. If the mains are not suitably covered with non-conducting material, their surface should be added to the radiating surface.

Example. — What should be the size of main to supply 400 feet of radiating surface, itself included? *Ans.* — $\sqrt{400} = 20$. Divide by 10, and we have 2 inches as the diameter of our main.

Return-pipes should be at least $\frac{3}{4}$ inch in diameter, and never less than one-half the diameter of the main, — longer returns requiring larger pipe. A thorough drainage of steam-pipes will effectually prevent all cracking and pounding noises therein.

Loss of Heat from Steam-Pipes.[1]

The following table shows the loss of heat from steam-pipes, naked, and clothed with wool or hair felt of different thicknesses. Steam pressure, 75 pounds. External air, 60°.

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[1] From Steam. Babcock & Wilcox Company, New York and Glasgow.

Thickness of Covering in Inches.	2 Inches Diameter. Loss in Units per Foot run per Hour.	Ratio of Loss.	Feet in Length per H.P. Lost.	4 Inches Diameter. Loss in Units per Foot run per Hour.	Ratio of Loss.	Feet in Length per H.P. Lost.	6 Inches Diameter. Loss in Units per Foot run per Hour.	Ratio of Loss.	Feet in Length per H.P. Lost.	8 Inches Diameter. Loss in Units per Foot run per Hour.	Ratio of Loss.	Feet in Length per H.P. Lost.	12 Inches Diameter. Loss in Units per Foot run per Hour.	Ratio of Loss.	Feet in Length per H.P. Lost.
	OUTSIDE DIAMETER OF PIPE, WITHOUT FELT.														
0	219.0	1.00	132	390.8	1.00	75			46	720.8	1.000	40	1077.4	1.000	26
$\frac{1}{4}$	100.7	0.46	288	180.9	0.46	160			–	–	–	–	–	–	–
$\frac{1}{2}$	65.7	0.30	441	117.2	0.30	247			154	219.6	0.301	132	301.7	0.280	92
1	45.8	0.20	662	73.4	0.18	392	111.0	0.178	261	128.3	0.176	225	185.3	0.172	157
2	28.4	0.13	1020	44.7	0.11	648	66.2	0.106	438	75.2	0.103	385	98.0	0.091	294
4	19.8	0.09	1464	28.1	0.07	1031	41.2	0.066	703	46.0	0.063	630	60.3	0.056	498
6	–	–	–	23.4	0.06	1238	33.7	0.054	860	34.5	0.047	845	45.2	0.042	642

There is a wide difference in the value of different substances for protecting from radiation, their value varying nearly in the inverse ratio of their conducting power for heat, up to their ability to transmit as much heat as the surface of the pipe will radiate, after which they become detrimental rather than useful as covering. This point is reached nearly at baked clay or brick.

Table of the conducting power of various substances, from Péclet:—

Substance.	Conducting Power.	Substance.	Conducting Power.
Blotting Paper . . .	0.274	Wood, across fibre . .	0.83
Eiderdown	0.314	Cork	1.15
Cotton or Wool, any density	0.323	Coke, Pulverized . .	1.29
		India Rubber	1.37
Hemp Canvas . . .	0.418	Wood, with fibre . .	1.40
Mahogany Dust . .	0.523	Plaster of Paris . . .	3.86
Wood Ashes	0.531	Baked Clay	4.83
Straw . . .	0.563	Glass	6.6
Charcoal Powder	0.636	Stone	13.68

A smooth or polished surface is of itself a good protection: polished tin or Russia iron having a ratio, for radiation, of 53 to 100 for cast iron. Mere color makes but little difference.

Hair or wool felt has the disadvantage of becoming soon charred from the heat of steam at high pressure, and sometimes of taking fire therefrom. This has led to a variety of "cements" for cover

ing pipes, composed generally of clay mixed with different substances, as asbestos, paper fibre, charcoal, etc. A series of careful experiments, made at the Massachusetts Institute of Technology in 1871, showed the condensation of steam in a pipe covered by one of them, as compared with a naked pipe and one clothed with hair felt, was 100 for the naked pipe, 67 for the "cement" covering, and 27 for the hair felt.

Table of relative value of non-conductors, from experiments by Charles E. Emery, Ph.D.: —

Non-Conductor.	Value.	Non-Conductor.	Value.
Wool-Felt	1.000	Loam, dry and open . .	0.550
Mineral Wool No. 2 . . .	0.832	Slacked Lime	0.480
" with Tar,	0.715	Gas-House Carbon . .	0.470
Sawdust	0.680	Asbestos	0.363
Mineral Wool No. 1 . . .	0.676	Coal Ashes	0.345
Charcoal	0.632	Coke in Lumps	0.277
Pine Wood, across fibre .	0.553	Air Space, undivided .	0.136

"Mineral wool," a fibrous material made from blast-furnace slag, is a good protection, and is incombustible.

Drying by Steam.[1]

There are three modes of drying by steam: 1st, by bringing wet substances in direct contact with steam-heated surfaces, as by passing cloth or paper over steam-heated cylinders, or clamping veneers between steam-heated plates; 2d, by radiated heat from steam-pipes, as in some lumber-kilns and laundry drying-rooms; 3d, by causing steam-heated air to pass over wet surfaces, as in glue-works, etc.

The second is rarely used except in combination with the third.

The first is most economical, the second less so, and the third least. Under favorable circumstances it may be estimated that one-horse power of steam will evaporate 24 pounds water by the first method, 20 by the second, and 15 by the third.

The philosophy of drying or evaporating moisture by heated air rests upon the fact that the capacity of air for moisture is rapidly increased by rise in temperature. If air at 52° is heated to 72°, its capacity for moisture is doubled, and is four times what it was at 32°. The following table gives the weight of a saturated mixture of air and aqueous vapor at different temperatures up to 160°, —

[1] From Steam. Babcock & Wilcox Company.

the practical limit of heating air by steam, — together with the weight of vapor, in pounds and percentage, and total heat, with the portion thereof contained in the vapor: —

SATURATED MIXTURES OF AIR AND AQUEOUS VAPOR.

Temperature, Degrees Fahrenheit.	Weight of 100 Cub. Ft. of Mixture in Lbs.	Weight of Water in 100 Cub. Ft. of Mixture in Lbs.	Per Cent of Water in Mixture.	Heat Units in 100 Cub. Ft. of Mixture.	Per Cent of Heat in Vapor.	Temperature, Degrees Fahrenheit.	Weight of 100 Cub. Ft. of Mixture in Lbs.	Weight of Water in 100 Cub. Ft. of Mixture, in Lbs.	Per Cent of Water in Mixture.	Heat Units in 100 Cub. Ft. of Mixture.	Per Cent of Heat in Vapor.
35	8.001	0.034	0.42	42.8	80.09	100	6.924	0.283	4.08	422.0	74.58
40	7.920	0.041	0.52	59.8	70.79	105	6.840	0.325	4.76	474.7	76.23
45	7.854	0.049	0.62	77.7	68.18	110	6.741	0.373	5.23	533.9	77.89
50	7.772	0.059	0.76	97.6	66.29	115	6.650	0.426	6.41	599.1	79.52
55	7.688	0.070	0.91	118.3	64.58	120	6.551	0.488	7.46	672.4	81.14
60	7.580	0.082	1.08	140.1	64.31	125	6.454	0.554	8.55	750.5	82.62
65	7.495	0.097	1.29	164.9	61.76	130	6.347	0.630	9.90	839.4	84.13
70	7.425	0.114	1.49	187.7	66.21	135	6.238	0.714	11.44	936.7	85.57
75	7.342	0.134	1.79	221.6	66.74	140	6.131	0.806	13.14	1042.7	86.90
80	7.262	0.156	2.15	254.6	68.02	145	6.015	0.909	15.11	1160.6	88.19
85	7.178	0.182	2.54	289.7	69.65	150	5.891	1.022	17.35	1288.4	89.39
90	7.108	0.212	2.98	330.2	71.19	155	5.764	1.145	19.88	1427.4	90.53
95	7.000	0.245	3.50	373.4	72.97	160	5.679	1.333	23.47	1608.7	91.93

By inspection of above table, it will be seen why it is more economical to dry at the higher temperatures. The atmosphere is seldom saturated with moisture, and in practice it will be found generally necessary to heat the air about 30° above the temperature of saturation. The best effect is produced where there is artificial ventilation, by fan or by chimney, and the course of the heated air is from above downwards.

Hot-Air, Steam, and Hot-Water Heating in Residences.

Much advancement has been made of late years in the methods of heating residences and in the apparatus intended for that purpose. While it is impossible in this book to treat the subject in detail, it is believed that the following information will be of value in deciding upon the kind of heating to be used, and in selecting an efficient apparatus, and seeing that it is properly put in.

In deciding upon a heating apparatus for a dwelling, the governing conditions are, generally, A, the size of the building, and, B, the limit of first cost. When the latter condition is not a controlling one, the cost of running the apparatus should be given the first consideration.

For residences of eight or ten rooms, and covering not more than 1,200 square feet of ground, the author would recommend hot-air heating by means of a good furnace.

For residences covering 1,400 square feet, a combination hot-air and water system is recommended, or an entire hot-water system.

For still larger residences, a steam or hot-water apparatus should be used.

Furnace Heating.—For warming residences not exceeding 1,200 square feet of ground area, the author believes a good furnace, properly set, and with hot-air pipes of proper size, suitably located, will give the best satisfaction, as it is economical in first cost, easy to manage, costs little for repairs, and furnishes a pleasant and healthy heat, at no greater expense of running than with steam or hot water.

The most common defects observed in furnace-heating are : Overheating of the air ; vitiating of the air by the gases of combustion ; and imperfect distribution of the heat.

The first two defects may be entirely avoided if sufficient care is exercised in the selection and setting-up of the furnace and in tending the fire, and the last defect may be reduced to a minimum by a wise location and proper proportion of the flues and registers.

The cause of the unsatisfactory heating of a great many houses, by furnaces, is in the owner or builder refusing to pay the necessary price for a first class furnace and for the best workmanship and materials. The same carelessness and "skinning" that is sometimes permitted with furnace-work, if permitted on a steam or hot-water apparatus, would in most cases prevent their working at all.

Furnace heating may be divided into two parts, the *production* of heat, and the *distribution* of the heat.

The former depends entirely upon the furnace, its setting, cold-air supply, draught, kind of fuel, and attendance.

The Furnace.—In principle, a hot-air furnace is simply a stove or heater, encased with iron or brick, so as to form an air chamber between the heater and casing. The air enters at the bottom of the chamber, passes over the heated surfaces of the heater, and is conducted by the hot-air pipes to the various rooms.

The external surface of the fire-pot, and all portions of the heater which receive heat from the fire or smoke, are called the *radiating* surface.

As a rule, the furnace which has the greatest radiating surface in proportion to the size of the fire-pot will give off the most heat for a given amount of fuel consumed.

As the amount of radiating surface largely affects the weight of a furnace, and the latter in a great measure the selling price, it is obvious that the best furnaces must cost the most. It is true that one furnace may have its radiating surfaces better arranged than another, so as to give off more heat for a less quantity of metal, but it is seldom that a very light furnace, particularly if of cast iron, is a good heater.

Furnaces should be so designed that the smoke, after leaving the combustion chamber, must travel around the radiator one or more times before finding an exit to the chimney. With a chimney flue of proper size and topped out well above the roof, it is possible to make the smoke travel a long distance, and thus obtain great economy of fuel. The best furnaces are designed on this principle.

Besides having large radiating surface, the furnace should have as few joints as possible, and should be arranged so as to be easily cleaned.

Furnaces are made of cast iron, wrought iron, and steel, either used singly or combined. The radiating surface above the fire-pot can be made more cheaply of wrought iron than of cast iron, and in certain arrangements it is just as serviceable.

While there are excellent furnaces made of wrought iron and steel, the author believes that a heavy cast iron furnace is the most durable, and can be made as tight. Some furnaces are made chiefly of cast iron, but with air or smoke flues of wrought iron fitting into cast iron sockets. This arrangement is not generally approved, as the two metals expand and contract unequally, thus tending to open the joints.

There are so many styles of furnaces manufactured that it is quite unadvisable to go further into details. It may be said, however, that the furnace shown in Fig. 1, made by the Richardson &

Boynton Company, is representative of the best type of cast-iron furnace, and that shown in Fig. 2, made by Isaac A. Sheppard & Co., a modern steel plate furnace. Fig. 3, of which the Excelsior Steel Furnace Company are the makers, shows a type of furnace which consists of a plain combustion chamber with a steel radiator. This radiator is divided with a horizontal partition, so that smoke must circulate entirely around it before it enters the flue. This furnace is intended for soft coal. The more modern furnaces, constructed for burning

Fig. 1.

soft coal, have provision for the introduction of superheated air into the fire-box, thereby preventing the formation of soot, and causing thorough combustion and intense heat. The one shown in Fig. 1 is a hot air blast furnace, and is supplied with oxygen at a high temperature for either hard or soft coal, accelerating and intensifying combustion to a very high degree. The Thatcher Furnace Company are makers of a tubular furnace that seems to possess considerable merit.

The casing surrounding the heater may be of brick or sheet iron. If of brick, it should consist of two four-inch walls with a space

The duct may be either carried horizontally under the basement ceiling until near the furnace, and then dropped to the air-pit, or it may be carried down against cellar wall, and thence under the floor to the furnace. The portion of the duct above the floor should be built of well seasoned, matched boards, or of galvanized iron. The portion below the floor should be constructed either of stone, brick, or glazed tile, and should be tightly cemented. If of brick

FIG. 3.

or stone, the duct should be covered with stone slabs, with the edges roughly dressed, and the joints cemented. The air-duct should not be carried under the floor if the soil is at all damp, nor near any drain.

Besides the external air supply, it is also a good idea to have a smaller air duct, leading from a register in the front hall to the base of the furnace. This duct may be of wood, tin, or galvanized iron, and may be connected either with the base of the furnace,

Ventilation.—A hot-air furnace plant, properly put in, will furnish a good supply of fresh air, and therefore afford fairly good ventilat on, if means are provided for carrying off the foul air in the rooms. The warm air entering a room must of necessity force out an equal quantity of the air already in the room; exits are often found in the spaces around the doors and windows, but these are rarely sufficient to carry away the air as fast as it would enter if unimpeded. Fireplaces, especially if kept in use, afford excellent ventilation. A good arrangement for obtaining ventilation is by building a large flue in a central chimney, and using a galvanized iron smoke-pipe, placed in the centre of it, for the furnace. The space surrounding the smoke-pipe may then be used for ventilation, and ducts from different rooms connected with it.

Location of Furnace.—Upon the location of the furnace the successful heating of the house often depends, and it is a matter that requires careful consideration.

As a general rule, the furnace should be located in the basement, near the centre of the space occupied by the registers, and a little nearer the side from which the prevailing winds come in winter-time. The tendency, in hot-air heating, when the wind is blowing strong in severe cold weather, is for the rooms on the further side of the house from the wind to be over-heated, while those against the wind are poorly heated, the registers on the windward side delivering almost no hot air. Therefore, to counteract this tendency, the furnace should be placed some few feet toward the windward side of the building, *provided* this does not make the pipes to the general, or family, living rooms longer than the others.

The height of the basement should be such that the "leaders," or horizontal hot-air pipes below basement ceiling, may have a pitch of one and one-half inches per running foot upward from the furnace. If there is no inclination to these pipes, the first-story rooms will be heated with difficulty. For a residence of ten rooms, the furnace-room should have a clear height of at least seven feet six inches.

Cold-Air Opening.—If only one external cold-air supply is used, it should be taken from the direction from which the prevailing winds come. For buildings in exposed situations it is desirable to have a cold-air supply from the opposite side of the building also, the ducts connecting, and each being furnished with a damper, so that either duct may be used, according to the direction of the wind. Cases have been known where the wind, blowing from the opposite direction of the cold-air supply, has sucked the air from the house, through the furnace and cold-air duct, thus actually reversing the

natural operation of the furnace. Two supplies will obviate this possibility.

Stacks and Registers.—To insure the best results, the location of furnace, stacks, and registers should be planned out before the work of construction begins, for while the building need not be planned to suit the heating apparatus, it almost always happens that the setting of the partitions, swinging of doors, and placing of studs and joists can be arranged so as to favor the placing of stacks and registers, without seriously affecting any desired arrangement of the plan, and this can be done much better on the plans than after the house is started.

It is generally conceded that the hot-air stacks should be placed in the partitions, and as near to the furnace as practicable, and that all horizontal branches should be as short as possible, as the air travels much slower in the horizontal branches, and more heat is lost from radiation. The registers should be placed as near the stack as possible; they should *not* be placed near the windows, nor where the doors will swing over or against them, nor in the floor near an open fireplace.

Whether the register shall be placed in the floor or partition, is a matter that should be decided by the owner. It is claimed that the circulation from a wall register is not as good as from one placed in the floor, and the wall above the register generally becomes discolored after a time, by the dust that is occasionally blown up through the pipes. On the other hand, floor registers catch much more dirt from sweeping the rooms, and many ladies object to having their carpets cut. The author believes that it is healthier to have the registers placed in the wall. Convex registers are to be preferred for walls, as they deliver more air than do the ordinary flat registers. It sometimes happens that the stacks must be put in an outside wall. When such is the case, the stack should be double, and wrapped with asbestos paper as well. Stacks should *not* be placed in outside walls, however, when it is possible to avoid it.

Calculations for Size of Furnace, Pipes, and Registers.

There appears to be no rule by which the architect can determine the size of furnace that should be used to heat a given building, other than by using the tables given by the various manufacturers. Rules have been given for determining the necessary grate area of

a furnace, but it is utterly impossible to make such a rule that will apply to all furnaces, as the heating capacity depends almost as much upon the amount and character of the radiating surface, and these vary with the make of the furnace. Some manufacturers give rules which take into account not only the cubic space to be heated, but also the outside wall and the glass area, both of which should be considered in deciding on the size of the heater. Most furnace-makers, however, merely give the amount of cubic space that the different sizes of their particular furnaces will heat, and as there is no way of telling how reliable these figures are, except by experience, it is wise to have the contractor give a guarantee that the furnace shall heat the building to 70° in zero weather without forcing the furnace.

Pipes and Registers.—The tables given in various books and catalogues for the size of pipes and registers vary a great deal, and must be used with considerable judgment. The following table appears to the author to be as reliable as any :

TABLE OF CAPACITY OF HOT-AIR PIPES AND REGISTERS.

Showing different sizes of hot-air registers used in furnace practice, together with the equivalents of the capacity of the same in round leader pipes from furnace, with elevation of at least one inch to the foot; also equivalent in riser pipes (or stacks), and also the cubic feet of space on first, second, and third floors which said registers with their proper round and square pipes will heat. This table is based on normal conditions, with runs of pipe of usual length, and is intended to show the size of registers and pipes necessary to raise the temperature of air from zero outside to 70° on the inside, within reasonable time, without forcing. The sizes that are marked with an asterisk are those recommended for general use. The larger the register the less resistance to the flow of the heated air, but sizes mentioned will produce good results, and, being stock sizes, will always be found in stock. In planning work arrange to use the sizes referred to.

Size of Register.	Equivalent in Round or Leader Pipe.	Equivalent in Square or Riser Pipe.	Cubic feet of space on first floor same will heat.	Cubic feet on second floor.	Cubic feet on third floor.
6 x 8	6 in.	4 x 8	400	450	500
*8 x 8	7 "	4 x 10	450	500	560
*8 x 10	8 "	4 x 10	500	850	880
*8 x 12	8 "	4 x 11	800	1000	1050
*9 x 12	9 "	4 x 12	1050	1250	1320
*9 x 14	9 "	4 x 14	1050	1350	1450
*10 x 12	10 "	4 x 14	1500	1650	1800
*10 x 14	10 "	6 x 10	1800	2000	2200
10 x 16	10 "	6 x 10	1800	2000	2200
12 x 14	12 "	6 x 12	2200	2300	2500
*12 x 15	12 "	6 x 12	2250	2300	2500
*12 x 17	12 "	6 x 14	2300	2500	2800
12 x 19	12 "	6 x 14	2300	2600	2800
*14 x 18	14 "	6 x 16	2800	3000	3200
*14 x 20	14 "	6 x 16	2900	3000	3200
*14 x 22	14 "	8 x 16	3000	3200	3400
*16 x 20	16 "	8 x 18	3600	4000	4250
*16 x 24	16 "	8 x 18	3700	4000	4250
*20 x 24	18 "	10 x 20	4800	5400	5750
*20 x 26	20 "	10 x 24	6000	7000	7450

It should always be borne in mind, however, that uniform heating does not depend so much upon the *actual* size of the pipes as upon the *relative* sizes. For example, in a two-story house of eight rooms of *exactly the same size*, and the same amount of wall and glass area, the best heating results will be obtained, not by using the same size of pipes for all the rooms, even if the pipes are of ample capacity, but by carefully proportioning the sizes of the pipes according to the exposure, length of the leaders, and whether the room is in the first or second story. The registers in the rooms with north and west exposures should be a little nearer the furnace, if possible, than the others, and the pipes to the first story should be larger than those leading to the second story.

Cold-Air Box.—The sectional area of the cold-air box should be equal to three-fourths of the aggregate sectional area of the leaders. The box, or duct, should be ten or twelve inches deep (for dwellings), and wide enough to give the required sectional area. It should also always be provided with a damper, so that the supply may be regulated to the heavy winds and extreme cold weather.

Specifications.

The following form is given as a guide to architects in preparing the specifications for furnace work :

SPECIFICATIONS FOR FURNACE WORK IN RESIDENCE, FOR MR. ...... TO BE BUILT AT ....

..................*Architect.*

Furnace.—Furnish and set up complete, where shown on basement plan, one No. — ——— furnace, portable pattern, with double casings. Connect the furnace with the chimney with No. 24 galvanized-iron smoke pipe, of the same size as the collar on the furnace; all bends or turns to be made with 3-piece elbows ; the pipe to be strongly supported by wire, and to be kept 12 ins. below the ceiling.

Air Pit.—Excavate for and build a cold-air chamber under the furnace, not less than 18 ins. deep, with 8-inch brick walls, laid and plastered with cement ; also cement the bottom of the chamber. Build the cold-air duct under cellar floor, where shown on plan, to be — ft. long, 14 ins. deep in the clear, and — ins wide, with sides of hard brick in cement, and the sides and bottom smoothly plastered with cement. Cover the duct with 3-inch flagstones with tight joints, leaving opening of proper size for the wooden box to be built by the carpenter (wooden box should be included in carpenter's specifications).

Hot-Air Pipes.—Furnish and properly connect with furnace, and register boxes, leaders and stacks of the following sizes, all to be made of bright IX tin, and the stacks to be double with air space between. All turns in leaders to be made by 3 or 4-piece elbows, and the stacks to have boots or starters of approved pattern.

SIZES OF PIPES AND REGISTERS.

Hall..........	12″ leader.	No stack.	12″ × 15″ register.
Parlor..........	10″ "	4″ × 14″ stack.	10″ × 12″ "
Dining-room....	12″ "	6″ × 12″ "	12″ × 15″ "
Library.........	10″ "	4″ × 14″ "	10″ × 12″ "
Chamber No. 1..	9″ "	4″ × 14″ "	9″ × 14″ "
" " 2..	9″ "	4″ × 12″ "	9″ × 12″ "
" " 3..	8″ "	4″ × 10″ "	8″ × 10″ "

Registers.—All registers are to be of sizes given in the foregoing list, of the Tuttle and Bailey manufacture, japanned, except those in the first story, which are to be electro bronze-plated. All

floor registers are to set in iron borders corresponding with the registers.

Register Boxes.—All register boxes to be made double; for first floor boxes the *joists are to be lined with tin* and provided with *ceiling plates* full size of register, with plaster collar attached, so that pipes and boxes can be removed without disturbing the plastering or defacing the ceiling.

Miscellaneous.—All horizontal pipes in the basement to be round, and where they pass through partitions they are to be provided with collars, so that the pipes can be removed without disturbing the plastering. All leaders to be provided with dampers and tin tags, designating the different rooms they supply; and, whenever pipes run near woodwork, the same is to be properly covered with tin, and protected from any danger from fire. The contractor is to remove all rubbish made by him, clean up all iron work, and leave the whole apparatus in complete working order, and furnish a poker of proper size.

Guarantee.—The contractor is to guarantee that the furnace shall, under proper management, heat all rooms with registers connected with the furnace to 70° Fahr. when temperature outside indicates 10° below zero. In event of the failure of the furnace to do this, the contractor is either to make the furnace heat said rooms or substitute another furnace that will heat the rooms, at his own expense, and without unnecessary delay.

Hot-Air and Water Combination.

It is quite difficult, if not impossible, to heat throughout dwellings covering more than 1,400 square feet, with warm air alone. On account of the much larger exposure and the increased length of leaders, it becomes necessary to supplement the warm air with an auxiliary heat which can be carried to remote and exposed parts of the house and which will not be affected by pressure of wind or long and crooked pipes. For supplying this auxiliary heat, hot water has been found best adapted, and a great variety of "combination" furnaces are now made which contain provisions for heating water which may be carried by pipes to radiators located in the portions of the house most difficult to heat by warm air. Such combination systems have been used with great success, and for heating dwellings of ten average-size rooms the author believes it to be the most successful system, as it guarantees the comfortable warming of the house, and, if properly put in, thorough ventilation, which cannot be obtained by any system of direct hot-water

or steam radiation. It is claimed that nearly 200 square feet of hot-water radiation can be obtained by absorbing the surplus heat which would usually be wasted in a warm-air furnace.

The construction of the parts for heating the water varies greatly with different makes of furnaces. Some furnaces have a portion of the fire-pot hollow, and the water is heated there ; others have a separate heater suspended over the fire-pot. It is impossible here to consider the relative merits of the various heaters ; the architect should examine the heaters for himself, and look up their record, before specifying any particular make.

As a rule, the portions of the house which should be heated by the hot water are the halls, bath-room, and perhaps the rooms on the north or west side of the house.

The same rules govern the size of the radiators and piping, and the manner of installing, as in an entire hot-water plant.

Hot-Water Heating.

Heating by hot water is regarded by many persons as the most nearly perfect method for heating residences. It certainly has many advantages, and there can be no question of the practicability of hot-water heating, particularly for residences.

Hot-water heating is accomplished by placing radiators either in the rooms to be heated, or in indirect stacks, the water being carried to and from them by a system of flow and return pipes. Beyond any little evaporation that may take place, the water is used continuously ; *i.e.*, it rises from the heater in the cellar to each of the radiators in the several rooms, the heat having been radiated through the surface of the pipes or sides of the radiators into the rooms, and the water having been cooled as it leaves the radiators passes through the return pipes to the base of the heater, through which, passing from the bottom to the top, it takes up the units of heat from the fire, and so passes again into the flow-pipes and on into the radiators as before, this circulation being continuous and its rapidity in exact proportion to the intensity of the fire.

A hot-water heating plant is very similar to a steam plant constructed on the two-pipe system, the principal difference being that no steam is generated, and the pipes and radiators are filled with water instead of steam.

Either the direct, direct-indirect, or indirect systems of radiation may be used, exactly the same as with steam ; and, aside from the boiler, or heater, there is no difference in the appearance of a steam and a hot-water apparatus.

The rooms are also heated in exactly the same way by both systems, viz., by radiation from the outside surface of the pipes and radiators; and the only difference there is between the two kinds of heat, as far as it affects the room, is that with hot-water heating the radiator is never heated above 200°, and seldom over 190°, so that the air cannot be overheated, as is often the case with steam. For this reason, and only this, hot-water heat is healthier than steam, when the latter is forced so as to keep the radiators at a very high temperature. The author believes, however, that too much stress is often laid on this point, and that practically there is little, if any, difference, as far as health is concerned, in the two kinds of heat.

The advantages of hot-water heating over steam, for residences, are:

1. Economy in running. Hot-water radiators will heat with a low fire, while with a steam apparatus no heat is given off unless the water is kept boiling. For dwellings this is a very important advantage, particularly in mild weather.

2. The heat of a hot-water apparatus can be perfectly controlled by either the fire in the heater, or the valve on the radiator, by partly closing it; whereas with steam radiators, the valve must be wide open or tightly closed. Also, with a hot-water apparatus some of the radiators may be run at their full capacity, while others may be partly or entirely shut off, without causing noise or in any way interfering with the perfect working of the system.

3. A hot water apparatus is perfectly noiseless in operation, there being none of the snapping or gurgling noises common with steam.

4. With hot-water heating there is no possible chance for an explosion, as the apparatus is open to the atmosphere through the expansion tank.

About the only objections that can be urged against hot water heating are: increased first cost, danger from freezing, extra space occupied by radiators, and the fact that a building cannot be as quickly warmed by hot water as by steam.

While in many buildings, especially those that are not kept warm all the time, many of these objections are of considerable importance, they do not, as a rule, hold good in residences, which are kept at a uniform temperature, and in which the extra size of the radiators is of little consequence.

In very cold weather, when the heating apparatus is worked to its full capacity, there is but little difference, if any, in the amount of coal consumed for either steam or hot-water heating.

The Heater.—When hot-water heating was first introduced, tubular boilers, similar to steam boilers, only entirely filled with tubes, were used for heating the water. Within the past ten years, however, a great many special heaters have been patented that are intended especially for residences, such as the "Gurney," "Mercer," "Gorton," and the "Furman." The American Boiler Company manufacture several, viz., the "Bolton," "Spence," "Tropic," "Perfect," and "Advance."

Nearly all of these heaters are made up of a number of cast-iron sections, which are bolted together and the joints packed to make them water-tight. The flow pipes are taken from the top of the upper section, and the return pipes are connected with the lowest section, which generally forms either the fire-pot or the ash-pit.

The successful working of a hot-water heating apparatus depends very largely upon the proper construction of the boiler. It is generally admitted that in an efficient hot-water heater the water must be cut up into small portions, so as to heat quickly, and the whole arrangement of the heater should be such that the least possible resistance is offered to free circulation.

The boiler in which the most powerful circulation is maintained with the least consumption of fuel is the most satisfactory as well as the cheapest.

The method employed in connecting the joints, and the facilities for cleaning fire surfaces, are also points that should be carefully examined.

For the efficiency of the various sizes and styles of heaters, the architect or owner must, as in the case of hot-air furnaces, depend largely upon the tables given by the manufacturers.

As there is no pressure on the heater other than the weight of the water, no steam-gauges, safety-valves, or similar appliances are required, as is the case with steam.

Radiation.—As has already been stated, the radiators and piping are practically the same for hot-water as for steam heat, except that, to heat a given space by hot-water circulation, more radiating surface is required than with steam.

The following ratio of heating surface to space heated is given by the Gurney Company, due allowance to be made for exposure, locality, glass surface, construction, and other conditions: *Dwellings:* One square foot of *direct* radiating surface will heat in parlor, sitting-room, living room, library, dining-room—from twenty-five to thirty-five cubic feet of air; hall, bath-room—from twenty to thirty cubic feet of air; sleeping rooms—from thirty to fifty cubic feet of air. For *indirect* radiation not less than fifty per

cent. more surface should be allowed, and for direct-indirect, twenty-five per cent. more.

Indirect Radiation.—Every residence heated, either by hot-water or steam radiation, should have at least two indirect radiators, to provide for some ventilation. These should be placed in the cellar, and connected with registers in the front hall and principal living room. The common method of providing for indirect radiation is explained on page 796.

Direct radiation, as has been explained elsewhere, simply heats the air in the room over and over, and not only does *not* afford any ventilation, but tends to decrease the vitalizing qualities of the air.

Expansion Tank.—Every job of hot-water heating (at least in residences) should have an open expansion tank, connected with the highest part of the flow-pipe. It should be placed in the bath-room or other convenient place, and not less than three feet above the highest radiator. The tank should be provided with a water-glass, to indicate the proper water level, which is usually about half-way up the glass. A one inch overflow pipe must also be provided, connected with tank about three or four inches from the top, and running to basement or other convenient place, where it will do no harm should the water in the expansion tank boil or overflow at any time. The expansion tank on a hot-water apparatus serves as a safety-valve. Should the water at any time be heated above the boiling-point, the steam finds its way through the flow-pipes to the tank, and thence escapes to the atmosphere. The expansion tank also allows the water in the system to expand or contract under different temperatures without any injury to the apparatus. The *capacity* of the expansion tank should be at least one-twentieth of the whole capacity of the apparatus.

A hot-water apparatus is generally filled by connecting the house supply to return pipe at or near the heater. Sometimes a supply is connected with the expansion tank, and a ball-cock placed on it, to insure that there shall always be three or four inches of water in the tank. At the lowest point of apparatus a draw-off, or emptying-cock, should be placed, to empty the system at any time.

The apparatus should be kept *full of water* during the summer months. This excludes the air, and prevents corrosion or oxidation of pipes.

Hot water heating requires a much more perfect apparatus than steam heating, and great care must be exercised in running and proportioning the flow and return pipes

The following *Advice to Fitters*, published by the Gurney Heat-

Manufacturing Company, contains many practical suggestions, that should be of almost equal interest to the architect and owner :

"When estimating upon a job, take well into consideration the extent of all flow, return pipes, and risers, also their situation, and calculate them as radiating surface in addition to what is placed in rooms, and allow *heater* power accordingly.

"Due care must be exercised to provide for any special conditions, such as exposure of building, material of construction, location, length and size of mains governing plant under consideration.

"Allowances should also be made for loose construction of doors and windows, which admit large volumes of cold air, and provide for outside doors which are used frequently, and open directly into the room.

"In estimating the radiating surface, it should be borne in mind that a large surface at a comparatively low temperature gives a much pleasanter atmosphere than a small surface at a high temperature.

"Excess of surface is no discomfort, as is the case with steam, since the temperature can easily be controlled by varying the fire, or by valve on radiator.

"All flow and return pipes in cellar should be properly covered with hair-felt or some other good non-conducting material, to obtain the best and most economical results. *Doing this will save one-sixth of the heat.* If no covering is used, paint heater and pipe exposed in basement a black or maroon japan ; neat and attractive piping goes far toward securing other contracts."

For a thorough and comprehensive treatise on hot-water heating and fitting, the reader is referred to a work on this subject by Mr. William J. Baldwin, published by the *Engineering Record.* Much valuable information may be found in the catalogues of the Gurney Heater Manufacturing Company, the H. B. Smith Company, and others.

Specification.

The following form may serve as a guide in specifying hot-water heating for residences :

SPECIFICATION FOR HOT-WATER HEATING APPARATUS, IN RESIDENCE FOR JOHN JONES, ESQ., BROOKLINE, MASS.

Heater.—Furnish and set up in cellar one No., (120 GURNEY) HOT-WATER HEATER, having fire, ash, and cleaning-out doors, shaking and sliding grate, with handle, draught dampers, and set of fire tools.

Make iron smoke connections to chimney; a flue of sufficient size to be furnished by the owner.

Trimmings.—Furnish all necessary trimmings, such as direct feed-cock, draw-off cock, for the purpose of filling and emptying the apparatus at any time.

The owner will furnish foundation to set heater upon in cellar, of proper size of base.

Pipes. Furnish and run all necessary flow and return pipes of ample size, connecting them to radiators with one-inch pipe (for each radiator) up to 52½ square feet of surface, and one and a quarter inch to radiators over 52½ square feet surface, and up to 120 square feet; over 120 square feet surface, one and one-half inch connections; said pipes to be of good and approved quality, one and one-half inch, and over, being lap-welded pipe.

Fittings. All fittings to be of gray iron, heavy pattern, full thread, and of good and approved quality; no malleable iron fittings to be used on the work.

All flow and return pipes in basement to be supported by neat, strong, and adjustable hangers, arranged to suit expansion and contraction, properly secured to timbers overhead.

At all points where pipes pass through ceilings, floors, or partitions, the channels or holes shall be protected with floor or ceiling plates.

Expansion Tank.—The expansion tank to be made of No. 22 galvanized iron, 25 inches high and 15 inches in diameter, and is to be furnished with a proper gauge glass, with brass mountings complete. It is to be placed above all the radiators, in some suitable place, and supported on a proper shelf. From this tank an overflow pipe will be run to basement or other suitable place.

Furnish and set up the following radiators, viz.:

	No. of Radiators.	Square Feet of Radiating Surface.
Main Hall, ...	1 Direct Radiator.	28 square feet.
	1 Indirect Radiator.	165 square feet.
Sitting room, ...	1 Direct Radiator.	72 square feet.
Library, ...	1 Direct Radiator.	40 square feet.
Dining room, ...	1 Direct Radiator.	60 square feet.
Sitting-room Chamber, . .	1 Direct Radiator.	40 square feet.
Library Chamber, ...	1 Direct Radiator.	42 square feet.
Dining-room Chamber, . .	1 Direct Radiator.	36 square feet.
Kitchen Chamber, . . .	1 Direct Radiator.	32 square feet.
Bathroom, ...	1 Direct Radiator.	30 square feet.
	10 Radiators.	545 square feet.

In all 380 square feet of direct surface and 165 square feet of indirect; total surface 545 square feet.

Each radiator to be supplied with a (Gurney) Radiator Valve, brass seat, full opening connected to flow pipe of radiator.

Each radiator is to have a neat, nickel-plated air-valve on its highest point, made to open and close with a key wrench.

All radiators and exposed pipes above cellar to be neatly bronzed in gold or silver bronze, or artistically painted, as chosen.

No carpenter's work included.

All pipes in basement to be covered with one inch hair felt, and neatly sewed up in canvas and painted one coat of good white lead.

The contractor to guarantee all materials and workmanship used in the construction of this apparatus to be the best of their respective kinds, and the apparatus to be complete, and capable of warming the rooms and halls in which radiators are placed to a temperature of — degrees Fahr., when the thermometer is at zero outside.

Steam Heating.

Although hot water is perhaps more popular just now for residence heating, there can be no question that a building can be as thoroughly warmed and ventilated by steam as by any other system, and generally at a smaller first cost. In very cold weather, it is doubtful if hot-water heating is as satisfactory as steam.

For indirect radiation, steam heat is generally considered cheaper than hot-water heat, and in every way as satisfactory.

For very large residences, the author would recommend steam heat, all of the principal rooms to be heated by indirect radiation, and only the bathroom, halls, and perhaps the attic and one or two rooms on the north side, which generally includes the dining-room, by direct radiation. For dining-rooms a special direct radiator, containing a warming closet, is made.

The air supply to the indirect stacks should be very large, and provided with a damper, so that the supply may be regulated according to the weather. If the indirect radiators are divided into sections, each section being controlled by a valve, either one-half, one-third, or the whole of the radiator may be used at will. The greater the radiation the more fuel will be consumed, and *vice versa*, so that when part of the radiation is cut off, the cost of running the boiler is reduced.

The same principles apply in heating a residence by steam as in heating any other building, and there is no difference in the piping and radiators. The boilers used in residence heating, however, are generally of a special pattern, designed especially for that class of work.

There is almost an infinite variety of these boilers, although a great many of them are of the same type. The requirements of an economical and satisfactory working boiler for house heating are as follows:

First.—They should contain a quantity of water sufficiently large to fill the pipes and radiators with steam to any required pressure *without lowering the water in the boiler to require an addition when steam is up;* for should the steam go down suddenly, there will be too much water in the boiler. This occurs in boilers made with very small parts, or pipes which have a small capacity at the water-line, and require great care; for should the boiler have an automatic water-feeder set for the *true* water line, it will fill up, but cannot discharge again when the steam goes down; while, if it has no feeder, there is danger of spoiling the boiler, as the water is in the pipes *in the form of steam.*

It is true that a boiler which contains a small amount of water in proportion to its heating surface will *get up steam* quicker than one containing a larger quantity of water, but the latter will keep steam much better when the fire is renewed; and boilers which contain small quantities of water are rapidly chilled as well as rapidly heated, and must be fired often and regularly.

Second.—The fire-box should be of iron, with a water space around it, to prevent clinkering on the sides, and the necessity of repairs to brickwork which are unavoidable in brick furnaces.

Third.—The fire-box should be *deep* below the fire-door, to admit of a thick fire to last all night, and thus keep up steam. For large boilers, which require the services of an engineer, it is desirable to have a large grate area and a thin fire; but such a fire requires to be renewed too often to be suitable for a house boiler.

Fourth.—The fire-box should be *spacious*, for the sake of good combustion.

Fifth.—The boiler should have few parts, and the *flues and tubes should be large* and in a vertical position, so that they will not foul easily, and that any deposit may fall to the bottom.

For dwellings, the writer advises those forms of boilers which are without tubes, or with but a very few, as the tubes will invariably give out long before the shell, and if the tubes are not kept clean they will transmit but a small percentage of heat.

Sixth.—All parts should be *readily accessible for cleaning and repairs.* This is a point of the greatest importance and economy. When the heating surfaces become covered with soot and ashes, the economy of the boiler greatly decreases, as the soot acts as an insulator and prevents the heat reaching the boiler. It is for this reason that boilers which work well when new are found insufficient to do the work required of them when they become dirty.

Seventh.—The heating surface should be arranged as nearly as possible at right angles to the currents of heated gases, and so break up the currents as to extract the entire available heat therefrom.

Eighth.—It should have, if possible, *no joints exposed to the direct action of the fire.*

Ninth.—It should have a great excess of strength over any legitimate strain, and should be so constructed as not to be liable to be strained by unequal expansion.

Tenth.—It should be durable in construction, and not liable to require early repairs.

Eleventh.—The water space should be divided into sections, so

arranged that should any section give out no general explosion can occur, and the destructive effects be confined to the simple escape of the contents.

Twelfth.—It should be proportioned for the work to be done, and be capable of working to its full rated capacity with the highest economy.

Thirteenth.—It should be provided with the very best gauges, safety-valves, and other fixtures.

The boiler should be set so that the water-line in the boiler will be at least four feet below the main horizontal supply-pipe.

FIG. 4.—SECTIONAL BOILER.

Sectional Boilers.—As there is always a possibility of an explosion in steam boilers it is desirable that in boilers intended for the heating of dwellings, and where no skilled engineer is employed, the danger from possible explosion shall be reduced to a minimum.

Safety from explosions is best obtained in a sectional boiler, which consists of a number of cast iron sections, placed side by side, and connected with each other by drums top and bottom.

A sectional boiler can perhaps be best described by Figs. 4 and 5, which show an outside view and a longitudinal section of the "Mercer" sectional boiler. As will be seen, it consists of a

number of cast-iron vertical sections set on a cast-iron base, which forms the ash-pit. Each section is a boiler in itself, and is connected with drums, top and bottom, arranged with nipple and lock-nut screw joints, as shown.

The front and rear sections form the front and rear of the boiler; the intermediate sections are all alike and as many of them as is necessary to do the required work may be used. In case one section of a boiler like this should become disabled, it will not generally do any great damage, and by cutting out the nipples and plugging the drums the boiler can be run for a time, until the broken section can be replaced.

FIG. 5. LONGITUDINAL SECTIONAL VIEW.

A boiler constructed similar to that shown in Figs. 4 and 5 is excellently adapted for house heating, by either steam or water, and as a rule they give good satisfaction.

There are several styles of sectional boilers manufactured, all being constructed on the same general principle.

There are other styles of house boilers that have given satisfaction, but the sectional boiler is probably the type most used. With respect to steam boilers for house heating, the "Mercer" and "Gold" sectional boilers, made by the H. B. Smith Company, the "Gurney," made by the Gurney Heater Manufacturing Company, the "Gorton," made by the Gorton & Lidgerwood Company,

the "American," made by the American Boiler Company, and the "Faultless Furman," produced by the Herendeen Manufacturing Company, of Geneva, N. Y., are among the best known, and are generally well liked. The "American" is simple in construction, and utilizes a large percentage of the products of combustion and generates steam quickly.

For burning soft coal the author believes that cast-iron sectional boilers will give the best satisfaction.

Typical Specification.

FOR A SUPERIOR LOW-PRESSURE STEAM-HEATING APPARATUS, FOR HEATING BY THE **INDIRECT** SYSTEM, WITH A STEAM PRESSURE OF FROM ONE TO FIVE POUNDS PER SQUARE INCH.

Boilers.—Furnish and erect in cellar, in position as shown on cellar plan, one (No. 4 Gorton Patent Side-Feed Boiler).

Fixtures.—Furnish and properly connect to said boiler the following improved attachments, viz.: One steam-gauge, one safety-valve, one water column, one glass water-gauge (with fixtures), three gauge-cocks, one automatic damper regulator, and all valves, pipes, and fittings necessary to render their connection to the boiler complete.

Furnish with said boiler the following fire tools, viz.: One hoe, one poker, and one slicing bar.

Connect the boiler to the chimney by means of a galvanized-iron smoke-pipe of suitable dimensions, with damper in same.

System of Piping.—This system of piping throughout will be constructed on the Double Pipe "Gravity Return" plan, and all pipes erected will be of ample size to insure the active delivery of dry steam to the radiators, and easy flow of the water of condensation back to the boiler.

Furnish and erect all supply and return mains and branch connecting-pipes of the sizes and located in the relative positions shown on the plans. All piping to be graded and properly dripped, and to be hung in position by means of expansion pipe-hangers.

Radiation.—The heating of the several apartments named will be accomplished by means of indirect radiators set in clusters or "stacks," each hung from and near the ceiling of the cellar, and the heat from these "stacks" will be conveyed to the room to be heated by means of tin hot-air pipes set in the walls and leading from cellar to the room to be heated; each room heated to have an independent "stack," and to be connected therewith by an independent tin hot-air pipe. Each of the "stacks" of indirect radiators will be inclosed in a neat and well-made box or casing made of galvanized iron, and from each "stack" there will be a galvanized-iron duct of proper size, leading to the nearest window, where the same shall be connected, to have opening to admit cold or fresh air to the "stack."

Radiators.—Furnish and erect in cellar, in the positions as shown on plans, ten "stacks" of approved pattern, indirect radiators, that in the aggregate will contain not less than 732 square feet of radiating surface, and divided up for the several rooms to be heated as follows, viz.:

First Story:

Hall,	1 "stack"	to	contain	108	sq.	ft.	sur.
Parlor,	1 "	"	"	96	"	"	"
Dining-room,	1 "	"	"	108	"	"	"
Library,	1 "	"	"	96	"	"	"
Rear hall,	1 "	"	"	48	"	"	"

Second Story:

Chamber over parlor,	1 "	"	"	72	"	"	"
" " dining-room,	1 "	"	"	72	"	"	"
" " library,	1 "	"	"	72	"	"	"
Hall bedroom,	1 "	"	"	36	"	"	"
Bathroom,	1 "	"	"	24	"	"	"

Valves.—The supply and the return connecting pipe to each "stack" will be provided with a globe valve, and each "stack" will have an approved automatic air-valve attached to it.

Pipe Covering.—All cellar pipes will be neatly covered with asbestos sheathing, then 1-inch-thick hair-felt and canvas casing sewed on.

Registers. Furnish and set in position, in each room heated, a vertical wheel register of the size shown on plans. All registers for first story to be bronze finish, and all others to be black or white japanned finish, as shall be selected.

Tin and Galvanized Iron Work.—Furnish to builder (and by him to be set in position as shown on plans) all tin wall-pipes for hot air to the rooms to be heated, all to be made of IX tin and of the sizes shown on plan.

Furnish and erect in cellar, as shown on plan, galvanized-iron casings or boxes for the ten "stacks," and to each "stack," from the nearest window, a galvanized iron duct to conduct fresh air to the "stacks;" all to be of the sizes and dimensions shown on plans, and to be constructed in a substantial and workmanlike manner; each fresh-air duct to be provided with a damper.

Quality of Materials.—All materials used in the construction of this apparatus are to be the best of their respective kinds; all fittings to be heavily beaded, and made of the best gray iron, with clean-cut threads.

Guarantee. The contractor is to guarantee that the apparatus when completed will be of ample capacity to maintain an even temperature of 70 degrees Fahrenheit in the rooms heated, when the outside temperature is zero; and that the apparatus will afford free circulation throughout, and be noiseless in operation.

Books on Residence Heating.—Much valuable information on residence heating may be obtained from pamphlets published by different manufacturers, among whom are the Gurney Heater Manufacturing Company, Gorton & Lidgerwood Company, Isaac A. Sheppard & Co., and the Excelsior Steel Furnace Company, of Chicago. The latter company publish a very complete book on furnace heating and furnace fittings, which every architect should have.

Temperature of Fire.

By reference to the table of fuels (p. 777), it will be seen that the temperature of the fire is nearly the same for all kinds of combustibles under similar conditions. If the temperature is known, the conditions of combustion may be inferred. The following table, from M. Pouillet, will enable the temperature to be judged by the appearance of the fire:—

Appearance.	Temperature F.	Appearance.	Temperature F.
Red, just visible	977°	Orange, deep	2010°
" dull	1290°	" clear	2190°
" Cherry, dull	1470°	White heat	2370°
" " full	1650°	" bright	2550°
" " clear . . .	1830°	" dazzling	2730°

To determine temperature by fusion of metals, etc.,—

Substance.	Temperature F.	Metal.	Temperature F.	Metal.	Temperature F.
Tallow,	92°	Bismuth,	518°	Silver, pure,	1830°
Spermaceti,	120°	Lead,	630°	Gold Coin,	2156°
Wax, white,	154°	Zinc,	793°	Iron, Cast, med.,	2010°
Sulphur,	239°	Antimony,	810°	Steel,	2550°
Tin,	455°	Brass,	1650°	Wrought-Iron,	2910°

Tables.

The following tables will be found useful in estimating the size of boilers, piping, registers, etc.

TABLE OF TEMPERATURE.

COMPILED FROM OBSERVATIONS OF THE SIGNAL SERVICE, U. S. A., AND BLODGETT'S CLIMATOLOGY OF THE UNITED STATES.

NOTE.—In the United States the comfortable temperature of the air in occupied rooms is generally 70 degrees, when walls have the same temperature.

STATION.	No. of mos. fire is required.	Mean temp. of fire mos.	Ave. no. of degrees temp. to be raised.	Max. no. degrees temp. to be raised.	Min. temp. of fire mos.
Albany, N. Y.	7	35	35	87	17
Baltimore, Md.	6	39	31	72	2
Boston, Mass.	7	37	33	81	11
Buffalo, N. Y.	8	35	35	83	13
Burlington, Vt.	7	32	38	90	20
Chicago, Ill.	7	35	35	90	20
Charleston, S. C.	3	52	18	47	23
Cincinnati, O.	7	42	28	77	7
Cleveland, O.	7	38	32	83	13
Detroit, Mich.	7	35	35	90	20
Duluth, Minn.	8	28	42	108	38
Indianapolis, Ind.	7	41	29	88	18
Key West, Fla.	0	0	0	26	44
Leavenworth, Kan.	6	37	33	90	20
Louisville, Ky.	6	42	28	80	10
Memphis, Tenn.	5	39	31	68	2
Milwaukee, Wis.	8	37	33	95	25
New Orleans, La.	0	0	0	44	26
New York, N. Y.	7	40	30	76	6
Philadelphia, Pa.	7	40	30	75	5
Pittsburg, Pa.	7	39	31	82	12
Portland, Me.	8	33	37	82	12
Portland, Ore.	6	43	27	67	3
San Francisco, Cal.	4	53	17	34	36
St. Louis, Mo.	5	37	33	86	16
St. Paul, Minn.	7	25	45	102	32
Washington, D. C.	5	40	30	73	3
Wilmington, N. C.	4	50	20	55	15

USEFUL MEMORANDA: HOT-WATER HEATING.

MEASUREMENT OF FLOW AND RETURN PIPES.

For the purpose of ascertaining the amount of heating surface in flow, return pipes, and risers, the following table is used. To obtain the surface, multiply length of pipe by figures given below, always pointing off two places.

Example : 500 lineal feet 1-inch pipe multiplied by .34 equals 170 square feet.

		Surface of Pipe covering ¾ inch Hair Felt and Canvas.		Table of Quantity of Water contained in 100 lineal feet of Pipe of different Diameters.	
Size of Pipe.	Square feet in one lineal foot.	Size of Pipe.	Multiply length by	Diameter of Pipe.	Contents in 100 feet in length.
¾ in.	.27	1 in.	.79	1 in.	4.50 gals.
1 in.	.34	1¼ in.	.96	1¼ in.	7.75 gals.
1¼ in.	.43	1½ in.	1.04	1½ in.	10.59 gals.
1½ in.	.50	2 in.	1.09	2 in.	17.43 gals.
2 in.	.62	2½ in.	1.20	2½ in.	24.80 gals.
2½ in.	.75	3 in.	1.37	3 in.	38.38 gals.
3 in.	.92	3½ in.	1.49	3½ in.	51.36 gals.
3½ in.	1.05	4 in.	1.64	4 in.	66.13 gals.
4 in.	1.17				

HORIZONTAL TUBULAR BOILERS.

MANUFACTURED BY KENDALL & ROBERTS, CAMBRIDGE, MASS.

					Horse-power.	Square feet of grate surface.	Lbs. of coal required per hour.	Square feet of radiating surface that can be supplied.
					52	62	558	14592
					51	58	522	13620
					42	54	486	12702
					34	50	450	12066
					25	48	432	11280
					17	44	396	10500
					08	40	360	9228
					06	36	324	8652
					72	28	252	6444
					82	32	288	7374
					77	30	270	6912
					72	28	252	6450
					67	26	234	5988
					60	26	234	5400
					65	26	234	[illegible]
					61	26	234	5502
					57	24	216	5142
					53	24	216	4782
					49	24	216	[illegible]
					46	20	180	4104
					43	18	152	3852
					40	18	152	3600
					37	18	152	3330
					34	18	152	3078
					36	18	152	3252
					34	16	144	3048
					32	16	144	2880
42	[illegible]	[illegible]			30	16	144	2646
42	13	38			27	14	120	2448
42	12	45			26	14	120	2340
42	11	45			24	14	[illegible]	2130
42	10	45			22	12	[illegible]	1920
42	9	45			19	12	108	1710
36	13	28	3	12	20	12	108	1836
36	12	34	2½	11	20	12	108	1788
36	11	34	2½	10	18	10	90	1620
36	10	34	2½	9	16	10	90	1464
36	9	34	2½	8	14	8	72	1296
36	8	31	2½	7	12	8	72	1140
30	9	30	2	8	10	6	54	912
30	8	30	2	7	8	6	54	768

Thickness of heads one eighth inch greater than thickness of shell.
The last three columns added by the author.

UPRIGHT TUBULAR BOILERS.

MANUFACTURED BY KENDALL & ROBERTS, CAMBRIDGEPORT, MASS.

Diameter of shell.	Height of shell.	Number of tubes.	Diameter of tubes.	Length of tubes.	Heating surface.	Horse-power.
ins.	ft. in.		in.	ft. in.	ft.	
18	4 0	40	1¼	0 33	–	–
18	4 6	40	1¼	0 39	–	–
18	5 0	40	1¼	0 45	–	–
24	5 0	25	2	3 6	52	3⅓
24	5 6	25	2	4 0	58	3⅔
24	6 0	25	2	4 6	64	4⅓
30	5 0	45	2	3 0	80	5
30	5 6	45	2	3 6	90	6
30	6 0	45	2	4 0	102	6⅔
30	6 6	45	2	4 6	114	7⅓
30	7 0	45	2	5 0	125	8⅓
36	6 0	65	2	4 0	145	9⅔
36	6 6	65	2	4 6	162	10⅔
36	7 0	65	2	5 0	180	12
36	7 6	65	2	5 6	195	13
36	8 0	65	2	6 0	210	14
42	6 6	100	2	4 6	240	16
42	7 0	100	2	5 0	268	18
42	7 6	100	2	5 6	293	19⅔
42	8 0	100	2	6 0	318	21
48	7 0	120	2	5 0	320	21
48	7 6	120	2	5 6	350	23
48	8 0	120	2	6 0	380	25
54	8 6	186	2	6 6	600	40
54	9 0	186	2	7 0	675	45
54	9 6	186	2	7 6	720	48
60	10 0	250	2	7 6	975	65
60	11 0	250	2	8 6	1100	73
60	12 0	250	2	9 6	1224	81

DIMENSIONS OF REGISTERS AND BORDERS.

MADE BY THE TUTTLE AND BAILEY Co.

DIMENSIONS OF REGISTERS AND BORDERS.—*Continued.*

ESTIMATED CAPACITY OF PIPES AND REGISTERS.

ROUND PIPES.

Diameter of pipe.	Area in sq. inches.	Diameter of pipe.	Area in sq. inches.	Diameter of pipe.	Area in sq. inches.
7 inches	38	12 inches.	113	22 inches.	380
8 "	50	14 "	154	24 "	452
9 "	63	16 "	201	26 "	531
10 "	78	18 "	254	28 "	616
11 "	95	20 "	314	30 "	707

RECTANGULAR PIPES.

Size of pipe.	Area in sq. inches.	Size of pipe.	Area in sq. inches.	Size of pipe.	Area in sq. inches.
4 × 8	32	8 × 20	160	12 × 18	216
4 × 10	40	8 × 24	192	12 × 20	240
4 × 12	48	10 × 12	120	12 × 24	288
4 × 16	64	10 × 15	150	14 × 14	196
6 × 10	60	10 × 16	160	14 × 16	224
6 × 12	72	10 × 18	180	14 × 20	280
6 × 16	96	10 × 20	200	16 × 16	256
8 × 10	80	12 × 12	144	16 × 18	288
8 × 12	96	12 × 15	180	16 × 20	320
8 × 16	128	12 × 16	192	16 × 24	384

REGISTERS.

Size of opening.	Capacity in sq. inches.	Size of opening.	Capacity in sq. inches.	Size of opening.	Capacity in sq. inches.
6 × 10	40	10 × 14	93	20 × 20	267
8 × 10	53	10 × 16	107	20 × 24	320
8 × 12	64	12 × 15	120	20 × 26	347
8 × 15	80	12 × 19	152	21 × 29	406
9 × 12	72	14 × 22	205	27 × 27	486
9 × 14	84	15 × 25	250	27 × 38	684
10 × 12	80	16 × 24	256	30 × 30	600

ROUND REGISTERS.

Size of opening.	Capacity in sq. inches.	Size of opening.	Capacity in sq. inches.	Size of opening.	Capacity in sq. inches.
7 inches.	26	12 inches.	75	20 inches.	209
8 "	33	14 "	103	24 "	301
9 "	42	16 "	134	30 "	471
10 "	52	18 "	169	36 "	679

TABLE OF SIZES AND DIMENSIONS OF SAFETY DOUBLE HOT-AIR STACKS,

Made by the Excelsior Steel Furnace Company.

INDEX TO ADDITIONS (SINCE NINTH EDITION).

INDEX.

DIMENSIONS OF REGISTERS AND BORDERS.

MADE BY THE TUTTLE AND BAILEY Co.

DIMENSIONS OF REGISTERS AND BORDERS.—*Continued*

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TABLE OF SIZES AND DIMENSIONS OF SAFETY DOUBLE HOT-AIR STACKS,

Made by the Excelsior Steel Furnace Company.

GLOSSARY

›f Technical Terms, Ancient and Modern, used by Architects, Builders, and Draughtsmen.

(*Compiled by the author from various sources.*)

Aaron's-Rod.—An ornamental figure representing a rod with a serpent wined about it. It is sometimes confounded with the caduceus of Mercury. 'he distinction between the caduceus and the Aaron's-rod is that the former as two serpents twined in opposite directions, while the latter has but one.

Abacus. The upper member of the capital of a column. It is sometimes quare and sometimes curved, forming on the plan egments of a circle called the arch of the abacus, nd is commonly decorated with a rose or other ornament in the centre, having the angles, called horns f the abacus, cut off in the direction of the radius r curve. In the Tuscan or Doric, it is a square ablet, in the Ionic, the edges are moulded, in the orinthian, its sides are concave and frequently nriched with carving. In Gothic pillars it has a reat variety of forms.

CORINTHIAN DORIC
ABACUS.

Abbey. A term for the church and other buildigs used by conventual bodies presided over by an bbot or abbess, in contradistinction to cathedral, which is presided over by a ishop, and priory, the head of which was a prior or prioress.

Abutment.—That part of a pier from which the arch springs.

Abuttals. The boundings of a piece of land on other land, street, river, etc.

Acanthus. A plant found in the south of Europe, representations of whose eaves are employed for decorating the Corinthian and omposite capitals. The leaves of the acanthus are sed on the bell of the capital, and distinguish the two ich orders from the three others

Acroteria.—The small pedestals placed on the exremities and apex of a pediment. They are usually ithout bases or plinths, and were originally intended › receive statues.

ACANTHUS.

Aile, Aisle.—The wings; inward side porticos of a church; the inward teral corridors which enclose the choir, the presbytery, and the body of the hurch along its sides. 2. Any one of the passages in a church or hall into which 1e pews or seats open

Alcove. The original and strict meaning of this word, which is derived from 1e Spanish *alcoba*, is confined to that part of a bed-chamber in which the bed ands, separated from the other parts of the room by columns or pilasters. It now commonly used to express any large recess in a room, generally sepaated by an arch.

Alipterion.—In ancient Roman architecture, a room used by bathers for nointing themselves.

Almonry.—The place or chamber where alms were distributed to the poor in churches, or other ecclesiastical building. At Bishopstone Church, Wiltshire, England, it is a sort of covered porch attached to the south transept, but not communicating with the interior of the church. At Worcester Cathedral, England, the alms are said to have been distributed on stone tables, on each side, within the great porch. In large monastic establishments, as at Westminster, it seems to have been a separate building of some importance, either joining the gate-house or near it, that the establishment might be disturbed as little as possible.

Altar.—In ancient Roman architecture, a place on which offerings or sacrifices were made to the gods. In Protestant churches, the communion table is often designated as the Altar, and in Roman Catholic churches it is a square table placed at the east end of the church for the celebration of mass.

Altar of Incense.—A small table covered with plates of gold on which was placed the smoking censer in the temple at Jerusalem.

Altar-piece.—The entire decorations of an altar; a painting placed behind an altar.

Altar-screen.—The back of the altar from which the canopy was suspended, and separating the choir from the lady chapel and presbytery. The Altar-screen was generally of stone, and composed of the richest tabernacle work of niches, finials, and pedestals, supporting statues of the tutelary saints.

Alto-rilievo.—High relief—a sculpture, the figures of which project from the surface on which they are carved.

Ambo. A raised platform, a pulpit, a reading-desk, a marble pulpit—an oblong enclosure in ancient churches, resembling in its uses and positions the modern choir.

Ambry. A cupboard or closet, frequently found near the altar in ancient churches to hold sacred utensils.

Ambulatory. An alley—a gallery—a cloister.

Amphiprostylos. A Grecian temple which has a columned portico on both ends.

Amphitheatre.—A double theatre, of an elliptical form on the plan, for the exhibition of the ancient gladiatorial fights and other shows. Its arena or pit, in which those exhibitions took place, was encompassed with seats rising above each other, and the exterior had the accommodation of porticos or arcades for the public.

Amphora. A Grecian vase with two handles, often seen on medals.

Ancones. The consoles or ornaments cut on the key-stones of arches or on the sides of door-cases. They are sometimes made use of to support busts or other figures.

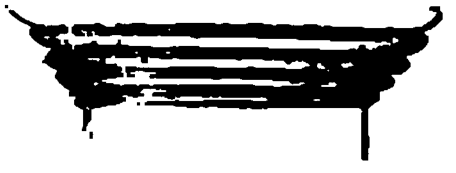
Angle-bar. In joinery, an upright bar at the angles of polygonal windows; a mullion.

Angle-capital. In Greek architecture, those Ionic capitals placed on the flank columns of a portico, which have one of their volutes placed horizontally at an angle of a hundred and thirty-five degrees with the plane of the frieze.

Annulated Columns. Columns clustered together by rings or bands; much used in English architecture.

Annular Vault. A vault rising from two parallel walls—the vault of a corridor. Same as *Barrel Vault*.

Annulet. A small square moulding used to separate others. The fillet which separates the flutings of columns is sometimes known by this term.



ANNULET.

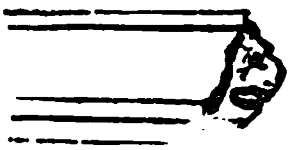
se.—A name given to a pilaster when attached to a wall. Vitruvius *parastatæ* when insulated. They are not usually diminished, and examples their capitals are different from those of the columns ny.

ber.—An apartment preceded by a vestibule and from which is nother room.

l.—A small chapel forming the entrance to another. There are Merton College, Oxford, and at King's College, Cambridge, England, al others. The antechapel to the lady-chapel in cathedrals is ed the Presbytery.

.—The part under the rood loft, between the doors of the choir entrance of the screen, forming a sort of lobby. It is also called r.

–In classical architecture (gargoyles, in Gothic architecture), the lions' and other heads below the eaves of a gh channels in which, usually by the mouth, carried from the eaves. By some this term is upright ornaments above the eaves in ancient which hid the ends of the Harmi or joint



ANTEFIXA.

.—The lowest part of the shaft of an Ionic or Corinthian column, t member of its base if the column be considered as a whole. The the inverted cavetto or concave sweep, on the upper edge of which ng shaft rests.

plain or moulded piece of finish below the stool of a window, put e rough edge of the plastering.

e semicircular or polygonal termination to the chancel of a church.

A temple without columns on the flanks or sides.

—An artificial canal for the conveyance of water, either above or . The Roman aqueducts are mostly of the former construction.

.—A building after the manner of the Arabs. Ornaments used by ple, in which no human or animal figures appear. sometimes improperly used to denote a species of or- posed of capricious fantastics and imaginary repre- animals and foliage so much employed by the Romans tions of walls and ceilings.

ARABESQUE.

rchitecture. A style of architecture the rudiments ear to have been taken from surrounding nations, the rians, Chaldeans, and Persians. The best preserved rtake chiefly of the Græco-Roman, Byzantine, and is supposed that they constructed many of their finest m the ruins of ancient cities.

.—That style of building in which the columns are one another from four to five diameters. Strictly term should be limited to intercolumniation of four ich is only suited to the Tuscan order.

los. That style of building in which four columns he space of eight diameters and a half; the central ation being three diameters and a half, and the others on each nly half a diameter, by which arrangement coupled columns are

-Large bronze candelabra, in the shape of a tree, placed on the floor urches, so as to appear growing out of it.

Arcade. A range of arches, supported either on columns or on piers, and detached or attached to the wall.

Arch. In building, a mechanical arrangement of building materials arranged in the form of a curve, which preserves a given form when resisting pressure, and enables them, supported by piers or abutments, to carry weights and resist pressure.

Arch-buttress. Sometimes called a flying buttress; an arch springing from a buttress or pier.

ARCADE.

Architrave. That part of an entablature which rests upon the capital of a column, and is beneath the frieze.

Architrave Cornice. An entablature consisting of an architrave and cornice, without the intervention of the frieze, sometimes introduced when inconvenient to give the entablature the usual height.

Architrave of a Door. The finished work surrounding the aperture; the upper part of the lintel is called the traverse; and the sides, the jambs.

Archives. A repository or closet for the preservation of writings or records.

Archivolt. A collection of members forming the inner contour of an arch, or a band or frame adorned with mouldings running over the faces of the arch-stones, and bearing upon the imposts.

Area. The superficial contents of any figure; an open space or court within a building; also an uncovered space surrounding the foundation walls to give light to the basement.

Arena. The plain space in the middle of the amphitheatre or other place of public resort.

Arris. The meeting of two surfaces producing an angle.

Arsenal. A public storehouse for arms and ammunition.

Artificer, or **Artisan.** A person who works with his hands, and manufactures any commodity in iron, brass, wood, etc.

Ashlar, or **Ashler.** A facing made of squared stones, or a facing made of thin slabs, used to cover walls of brick or rubble. *Coursed ashlar* is where the stones run in level courses all around the building; *random ashlar*, where the stones are of different heights, but level beds. 2. Common freestones of small size, as they come from the quarry, are also called ashlar.

Asphaltum. A kind of bituminous stone, principally found in the province of Neufchatel. Mixed with stone, it forms an excellent cement, incorruptible by air and impenetrable by water.

Astragal. A small semicircular moulding, sometimes plain and sometimes ornamental.

Asymptote. A straight line which continually approaches to a curve without touching it.

Atlases, or **Atlantes.** Figures or half-figures of men used instead of columns or pilasters to support an entablature; called also Telamones.

Atrium. A court in the interior division of Roman houses.

Attached Columns. Those which project three-fourths of their diameter from the wall.

Attic. A low story above an entablature, or above a cornice which limits the height of the main part of an elevation. Although the term is

ATLANTES.

ved from the Greek, we find nothing exactly answering to it in cture; but it is very common in both Roman and Italian practice. erwise called tholobates in St. Peter's and St. Paul's Cathedrals termed attics.

ır.—A term used to denote the low pilasters employed in the an attic story

.—In painting and sculpture, symbols given to figures and statues ıir office and character.

-In ancient churches, that part of the church where the people to be instructed in the Gospel, now called the nave.

ourt or hall in ancient Roman houses.

. large apartment for breeding birds.

spindle or centre of any rotative motion. In a sphere, an imagough the centre.

.—A place behind the altar in the principal choir, in which there nall altar standing back to back with the former.

f a Rafter or Rib. The forming of an upper or outer surface, ıge with the edges of the ribs or rafters on either side.

'a Wall.—The rough inner face of a wall; earth deposited behind .il, etc.

Window.—That piece of wainscoting which is between the bottom ıme and the floor

A projection from the face of a wall, supported by columns or conılly surrounded by a balustrade.

—A building in the form of a canopy, supported with columns, ı a crown or covering to an altar.

A small pillar or column, supporting a rail, ns, used in balustrades

ıaft.—The shaft dividing a window in Saxon At St. Albans are some of these shafts, evi-.he old Saxon church, which have been fixed an capitals.

ı.—A series of balusters connected by a rail. ort of flat frieze or fascia running horizon- tower or other parts of a building, particu-tables in perpendicular work, commonly used shafts characteristic of the thirteenth cen-rally has a bold, projecting moulding above ıd is carved sometimes with foliages, but in cusped circles, or quatrefoils, in which frequently are shields of

BALDACHIN.

Column. A series of annulets and hollows going round the middle of columns, and sometimes of the entire pier. They are often beau-with foliages, etc., as at Amiens. In several cathedrals there are se apparently covering the junction of the frusta of the columns, and Westminster they appear to have been gilt; they are there called Shaft-rings.

.—A separate building to contain the font, for the rite of baptism. nent on the Continent; that at Rome, near St. John Lateran, and nce, Pisa, Pavia, etc., are all well-known examples. The only exgland are at Cranbrook and Canterbury; the latter, however, is ave been originally part of the treasury.

Barbican.—An outwork for the defence of a gate or drawbridge; also, a sort of pent house or construction of timber to shelter warders or sentries from arrows or other missiles.

Barge Board.—See *Verge Board*.

Bartizan.—A small turret, corbelled out at the angle of a wall or tower, to protect a warder and enable him to see around him. They generally are furnished with oylets or arrow-slits.

BARTIZAN.

Basement.—The lower part of a building, usually in part below the grade of the lot or street.

Base Mouldings.—The mouldings immediately above the plinth of a wall, pillar, or pedestal.

Base of a Column.—That part which is between the shaft and the pedestal, or, if there be no pedestal, between the shaft and the plinth. The Grecian Doric had no base, and the Tuscan has only a single torus, or a plinth.

Basilica.—A term given by the Greeks and Romans to the public buildings devoted to judicial purposes.

Bas-relief.—See *Basso-rilievo*.

Basse-cour.—A court separated from the principal one, and destined for stables, etc.

Basso-rilievo, or **Bas-relief.**—The representations of figures projected from a background without being detached from it. It is divided into three parts: Alto-rilievo, when the figure projects more than one-half; Mezzo-rilievo, that in which the figure projects one half; and Basso-rilievo, when the projection of the figure is less than one-half, as in coins.

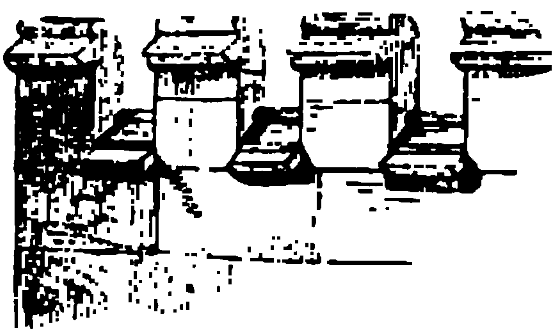
Bat.—A part of a brick.

Batten.—Small scantlings, or small strips of boards, used for various purposes. 2. Small strips put over the joints of sheathing to keep out the weather.

Batten-door.—A door made of sheathing, secured by strips of board, put crossways, and nailed with clinched nails.

Batter.—A term used by bricklayers, carpenters, etc., to signify a wall, piece of timber, or other material, which does not stand upright, but inclines from you when you stand before it; but when, on the contrary, it leans toward you, it is said to overhang.

Battlement.—A parapet with a series of notches in it, from which arrows may be shot, or other instruments of defence hurled on besiegers. The raised portions are called merlons; and the notches, embrasures or crenelles. The former were intended to cover the soldier while discharging his weapon through the latter. Their use is of great antiquity; they are found in the sculptures of Nineveh, in the tombs of Egypt, and on the famous François vase, where there is a delineation of the siege of Troy. In ecclesiastical architecture the early battlements have small shallow embrasures at some distance apart. In the Decorated period they are closer together, and deeper, and the mouldings on the top of the merlon and bottom of the embrasure are richer. During this period, and the early part of the Perpendicular, the sides or cheeks of the embrasures are perfectly square and plain. In later times the mouldings were continued round the sides, as well as at top and bottom, mitring at the angles, as over the doorway of Magdalene Col-



BATTLEMENT.

lege, Oxford, England. The battlements of the Decorated and later periods are often richly ornamented by panelling, as in the last example. In castellated work the merlons are often pierced by narrow arrow-slits. (See Oylet.) In South Italy some battlements are found strongly resembling those of old Rome and Pompeii; in the Continental ecclesiastical architecture, the parapets are very rarely embattled.

Bay.—Any division or compartment of an arcade, roof, etc. Thus each space, from pillar to pillar, in a cathedral, is called a bay, or severy.

Bay Window.—Any window projecting outward from the wall of a building, either square or polygonal on plan, and commencing from the ground. If they are carried on projecting corbels, they are called Oriel windows. Their use seems to have been confined to the later periods. In the Tudor and Elizabethan styles they are often semicircular in plan, in which case some think it more correct to call them Bow Windows.

Bazaar.—A kind of Eastern mart, of Arabic origin.

Bead.—A circular moulding. When several are joined, it is called Reeding; when flush with the surface, it is called Quirk-bead; and when raised, Cock-bead.

Beam.—A piece of timber, iron, stone, or other material, placed horizontally, or nearly so, to support a load over an opening, or from post to post.

Bearing.—The portion of a beam, truss, etc., that rests on the supports.

Bearing Wall, or **Partition.**—A wall which supports the floors and roofs in a building.

Beaufet, or **Buffet.**—A small cupboard, or cabinet, to contain china. It may either be built into a wall, or be a separate piece of furniture.

Bed.—In bricklaying and masonry, the horizontal surfaces on which the stones or bricks of walls lie in courses.

Bed of a Slate.—The lower side.

Bed Mouldings.—Those mouldings in all the orders between the corona and frieze.

Belfry.—Properly speaking, a detached tower or campanile containing bells, as at Evesham, England, but more generally applied to the ringing-room or loft of the tower of a church. See *Tower*.

Bell-cot, Bell-gable, or **Bell-turret.**—The place where one or more bells are hung in chapels, or small churches which have no towers. Bell-cots are sometimes double, as at Northborough and Coxwell, England; a very common form in France and Switzerland admits of three bells. In these countries, also, they are frequently of wood, and attached to the ridge. Those which stand on the gable, dividing the nave from the chancel, are generally called Sanctus Bells. A very curious and, it is believed, unique example at Cleves Abbey, England, juts out from the wall. In later times bell-turrets were much ornamented; these are often called Flèches.

Bell of a Capital.—In Gothic work, immediately above the necking is a deep, hollow curve; this is called the bell of a capital. It is often enriched with foliages. It is also applied to the body of the Corinthian and Composite capitals.

Belt.—A course of stones or brick projecting from a brick or stone wall, generally placed in a line with the sills of the windows; it is either moulded, fluted, plane, or enriched with patras at regular intervals. Sometimes called Stone String.

Belvedere, or **Look-out.**—A turret or lantern raised above the roof of an observatory for the purpose of enjoying a fine prospect.

Bema.—The semicircular recess, or hexedra, in the basilica, where the judges sat, and where in after-times the altar was placed. It generally is roofed with a half-dome or concha. The seats of the priests were against the wall, looking

into the body of the church, that of the bishop being in the centre. The bema is generally ascended by steps, and railed off by cancelli.

Bench Table.—The stone seat which runs round the walls of large churches, and sometimes round the piers; it very generally is placed in the porches.

Bevel.—An instrument for taking angles. One side of a solid body is said to be bevelled with respect to another, when the angle contained between those two sides is greater or less than a right angle.

Bezantee. A name given to an ornamented moulding much used in the Norman period, resembling bezants, coins struck in Byzantium.

Billet. A species of ornamented moulding much used in Norman, and sometimes in Early English work, like short pieces of stick cut off and arranged alternately.

Blocking, or Blocking-course.—In masonry, a course of stones placed on the top of a cornice crowning the walls.

Bond. In bricklaying and masonry, that connection between bricks or stones formed by lapping them upon one another in carrying up the work, so as to form an inseparable mass of building, by preventing the vertical joints falling over each other. In brickwork there are several kinds of bond. In common brick walls in every sixth or seventh course the bricks are laid crossways of the wall, called Headers. In face work, the back of the face brick are clipped so as to get in a diagonal course of headers behind. In Old English bond, every alternate course is a header course. In Flemish bond, a header and stretcher alternate in each course.

Bond-stones. Stones running through the thickness of the wall at right angles to its face, in order to bind it together.

Bond-timbers. Timbers placed in a horizontal direction in the walls of a brick building in tiers, and to which the battens, laths, etc., are secured. In rubble work, walls are better plugged for this purpose.

Border. Useful ornamental pieces around the edge of anything.

Boss. An ornament, generally carved, forming the key-stone at the intersection of the ribs of a groined vault. Early Norman vaults have no bosses. The carving is generally foliage, and resembles that of the period in capitals, etc. Sometimes they have human heads, as at Notre Dame at Paris, and sometimes grotesque figures. In Later Gothic vaulting there are bosses at every intersection.

Boutell. The mediæval term for a round moulding, or torus. When it follows a curve, as round a bench end, it is called a Roving Boutell.

Bow. Any projecting part of a building in the form of an arc of a circle. A bow, however, is sometimes polygonal.

Bow Window. A window placed in the bow of a building.

Brace. In carpentry, an inclined piece of timber, used in trussed partitions, or in framed roofs, in order to form a triangle, and thereby stiffen the framing. When a brace is used by way of support to a rafter, it is called a strut. Braces in partitions and span-roofs are, or always should be, disposed in pairs, and introduced in opposite directions.

Brace Mould. { } Two ressaunts or ogees united together like a brace in printing, sometimes with a small bead between them.

Bracket. A projecting ornament carrying a cornice. Those which support vaulting-shafts or cross springers of a roof are more generally called Corbels.

Break. Any projection from the general surface of a building.

Breaking Joint. The arrangement of stones or bricks so as not to allow two joints to come immediately over each other. See *Bond*.

Breast of a Window. The masonry forming the back of the recess and the parapet under the window-sill.

Bressummer.—A lintel, beam, or iron tie, intended to carry an external wall and itself supported by piers or posts; used principally over shop windows. This term is now seldom used, the word *beam*, or *girder*, taking its place.

Bridging.—A method of stiffening floor joist and partition studs, by cutting pieces in between Cross bridging of floor joist is illustrated in cut.

BRIDGES.

Bulwark.—In ancient fortification, nearly the same as Bastion in modern.

Burse, or **Bourse.**—A public edifice for the assembly of merchant traders; an exchange.

Bust. In sculpture, that portion of the human figure which comprises the head, neck, and shoulders.

Buttery.—A store-room for provisions.

Butt-joint.—Where the ends of two pieces of timber or moulding butt together.

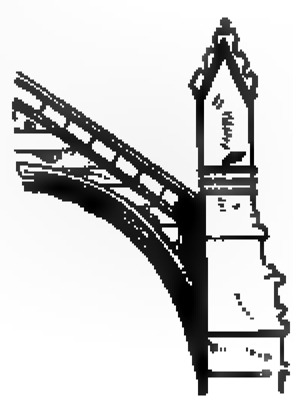
Buttress.—Masonry projecting from a wall, and intended to strengthen the same against the thrust of a roof or vault. Buttresses are no doubt derived from the classic pilasters which serve to strengthen walls where there is a pressure of a girder or roof-timber. In very early work they have little projection, and, in fact, are "strippilasters." In Norman work they are wider, with very little projection, and generally stop under a cornice or corbel table. Early English buttresses project considerably, sometimes with deep sloping weatherings in several stages, and sometimes with gabled heads. Sometimes they are chamfered, and sometimes the angles have jamb shafts. At Wells and Salisbury, England, they are richly ornamented with canopies and statues. In the Decorated period they became richly panelled in stages, and often finish with niches and statues and elegantly carved and crocketed gabelts, as at York, England. In the Perpendicular period the weatherings became waved, and they frequently terminate with niches and pinnacles.

BUTTRESS.

Buttress, Flying.—A detached buttress or pier of masonry at some distance from a wall, and connected therewith by an arch or portion of an arch, so as to discharge the thrust of a roof or vault on some strong point.

Buttress Shafts. Slender columns at the angle of buttresses, chiefly used in the Early English period.

Byzantine Architecture. A style developed in the Byzantine Empire. The capitals of the pillars are of endless variety and full of invention; some are founded on the Greek Corinthian, some resemble the Norman and the Lombard style, and so varied that no two sides of the same capital are alike. They are comprised under the style Romanesque, which comprehends the round-arch style. Byzantine architecture reached its height in the Church of St. Sophia at Constantinople.



FLYING BUTTRESS.

Cabinet. A highly ornamented kind of buffet or chest of drawers set apart for the preservation of things of value.

Cabling.—The flutes of columns are said to be cabled when they are partly occupied by solid convex masses, or appear to be refilled with cylinders after they had been formed.

Caduceus.—Mercury's rod, a wand entwined by two serpents and surmounted by two wings. The rod represents power; the serpents, wisdom; and the wings, diligence and activity.

CADUCEUS.

Caisson.—A panel sunk below the surface in flat or vaulted ceilings. See *Casson*.

Caisson.—In bridge building, a chest or vessel in which the piers of a bridge are built, gradually sinking as the work advances till its bottom comes in contact with the bed of the river, and then the sides are disengaged, being so constructed as to allow of their being thus detached without injury to its floor or bottom.

Caliber, or **Caliper.**—The diameter of any round body; the width of the mouth of a piece of ordnance.

Camber.—In carpentry, the convexity of a beam upon the surface, in order to prevent its becoming concave by its own weight, or by the burden it may have to sustain.

Campanile.—A name given in Italy to the bell-tower of a town-hall or church In that country this is almost always detached from the latter.

Candelabrum.—Stand or support on which the ancients placed their lamps. Candelabra were made in a variety of shapes and with much taste and elegance. The term is also used to denote a tall ornamental candlestick with several arms, or a bracket with arms for candles.

Canopy.—The upper part or cover of a niche, or the projection or ornament over an altar, seat, or tomb. The word is supposed to be derived from conopæum, the gauze covering over a bed to keep off the gnats; a mosquito curtain. Early English canopies are generally simple, with trefoiled or cinque-foiled heads; but in the later styles they are very rich, and divided into compartments with pendants, knots, pinnacles, etc. The triangular arrangement over an Early English and Decorated doorway is often called a canopy. The triangular canopies in the North of Italy are peculiar. Those in England are generally part of the arrangement of the arch mouldings of the door, and form, as it were, the hood-moulds to them, as at York. The former are above and independent of the door mouldings, and frequently support an arch with a tympanum, above which is a triangular canopy, as in the Duomo at Florence. Sometimes the canopy and arch project from the wall, and are carried on small jamb shafts, as at San Pietro Martiro at Verona. Canopies are often used over windows, as at York Minster over the great west window, and lower tiers in the towers. These are triangular, while the upper windows in the towers have ogee canopies.

Capital.—The upper part of a column, pilaster, pier, etc. Capitals have been used in every style down to the present time. That mostly used by the Egyptians was bell-shaped, with or without ornaments. The Persians used the double-headed bell, forming a kind of bracket capital. The Assyrians apparently made use of the Ionic and Corinthian, which were developed by the Greeks, Romans, and Italians into their present well-known forms. The Doric was apparently an invention or adaptation by the Greeks, and was altered by the Romans and Italians. But in all these examples, both ancient and modern, the capitals of an order are all of the same form throughout the same building, so that if one be seen the form of all the others is known. The Romanesque architects altered all this, and in the carving of their capitals often introduced such figures and emblems as helped to tell the story of their building. Another form was introduced by them in the curtain capital, rude at first, but afterward highly decorated. It evidently took its origin from the cutting off of the lower angles of a square block, and then rounding them off. The process may be distinctly seen, in its several stages, in Mayence Cathedral. But this form of capital was more

fully developed by the Normans, with whom it became a marked feature. In the early English capitals a peculiar flower of three or more lobes was used spreading from the necking upward in most graceful forms. In Decorated and Perpendicular styles this was abandoned in favor of more realistic forms of crumpled leaves, enclosing the bell like a wreath. In each style bold abacus mouldings were always used, whether with or without foliage.

Caravansary.—A huge, square building, or inn, in the East, for the reception of travellers and lodging of caravans.

Carriage.—The timber or iron joist which supports the steps of a wooden stair.

Carton, or **Cartoon.**—A design made on strong paper, to be transferred on the fresh plaster wall to be afterward painted in fresco; also, a colored design for working in mosaic tapestry.

Cartouche.—An ornament resembling a scroll of paper, being usually in form of a table, or flat member, with wavings, bearing some inscription or device. It is nearly akin to a modillion, with this exception, that the cartouche is used only externally, while the modillion is used both internally and externally, as under the cornice in the eaves of a house.

Caryatides.—Human female figures used as piers, columns, or supports. *Caryatic* is applied to the human figure generally, when used in the manner of caryatides.


CARYATID.

Cased.—Covered with other materials, generally of a better quality.

Casement.—A glass frame which is made to open by turning on hinges affixed to its vertical edges.

Cassoon, or **Caisson.**—A deep panel or coffer in a soffit or ceiling. This term is sometimes written in the French form, *caisson*; sometimes derived directly from the Italian *cassone*, the augmentative of *cassa*, a chest or coffer.

Cast.—A term used in sculpture for the impression of any figure taken in plaster of Paris, wax, or other substances.

Catacombs.—Subterranean places for burying the dead. Those of Egypt, and near Rome, are believed to be the most important.

Catafalco.—An ornamental scaffold used in funeral solemnities.

Cathedral.—The principal church, where the bishop has his seat as diocesan.

Cauliculus.—The inner scroll of the Corinthian capital. It is not uncommon, however, to apply this term to the larger scrolls or volutes also.

Causeway.—A raised or paved way.

Cavetto.—A concave ornamental moulding, opposed in effect to the ovolo—the quadrant of a circle.

Ceiling.—That covering of a room which hides the joists of the floor above, or the rafters of the roof. Most European churches have either open roofs, or are groined in stone. At Peterborough and St. Albans, England, there are very old flat ceilings of boards curiously painted. In later times the boarded ceilings, and, in fact, some of those of plaster, have moulded ribs, locked with bosses at the intersection, and are sometimes elaborately carved. In many English churches there are ceilings formed of oak ribs, filled in at the spandrels with narrow, thin pieces of board, in exact imitation of stone groining. In the Elizabethan and subsequent periods the ceilings are enriched with most elaborate ornaments in stucco. 2. Matched and beaded boards, planed and smoothed, used for wainscoting. In the New England States it is called sheathing.

Cenotaph.—An honorary tomb or monument, distinguished from monuments in being empty, the individual it is to memorialize having received interment elsewhere.

Centaur.—A poetical imaginary being of heathen mythology, half-man and half-horse.

Centring.—In building, the frames on which an arch is turned.

Chamfer, Champfer, or **Chaumfer.**—When the edge or arris of any work is cut off at an angle of 45° in a small degree, it is said to be chamfered; if to a large scale, it is said to be a canted corner. The chamfer is much used in mediæval work, and is sometimes plain, sometimes hollowed out, and sometimes moulded.

Chamfer Stop.—Chamfers sometimes simply run into the arris by a plane face; more commonly they are first stopped by some ornament, as by a bead; they are sometimes terminated by trefoils, or cinque-foils, double or single, and in general form very pleasing features in mediæval architecture.

Chancel.—A place separated from the rest of a church by a screen. The word is now generally used to signify the portion of an Episcopal or Catholic church containing the altar and communion table.

Chantry.—A small chapel, generally built out from a church. They generally contain a founder's tomb, and are often endowed places where masses might be said for his soul. The officiator, or mass priest, being often unconnected with the parochial clergy; the chantry has generally an entrance from the outside.

Chapel.—A small, detached building used as a substitute for a church in a large parish; an apartment in any large building, a palace, a nobleman's house, a hospital or prison, used for public worship; or an attached building running out of and forming part of a large church, generally dedicated to different saints, each having its own altar, piscina, etc., and screened off from the body of the building.

Chapter House.—The chamber in which the chapter or heads of the monastic bodies assembled to transact business. They are of various forms; some are oblong apartments, some octagonal, and some circular.

Chaptrel.—In Gothic architecture, the capital of a pier or column which receives an arch.

Charnel House.—A place for depositing the bones which might be thrown up in digging graves. Sometimes it was a portion of the crypt; sometimes it was a separate building in the church-yard; sometimes chantry chapels were attached to these buildings. M. Viollet-le-Duc has given two very curious examples of *ossuaires*—one from Fleurance, the other from Faouet.

CHAPTREL.

Cherub Gothic.—A representation of an infant's head joined to two wings, used in the churches on key-stones of arches and corbels.

Chevron Gothic.—An ornament turning this and that way, like a zigzag, or letter Z.

CHEVRON.

Chiaro-oscuro.—The effects of light and shade in a picture.

Choir.—That part of a church or monastery where the breviary service, or "horæ," is chanted.

Church.—A building for the performance of public worship. The first churches were built on the plan of the ancient basilicæ, and afterward on the plan of a cross; a church is said to be in Greek cross when the length of the transverse is equal to that of the nave; in Latin cross, when the nave is longer than the transverse part; in rotundo, when it is a perfect circle; simple, when it has only a nave and choir; with aisles, when it has a row of porticos in form of vaulted galleries, with chapels in its circumference.

Ciborium.—A tabernacle or vaulted canopy supported on shafts standing over the high altar.

Cincture.—A ring, list, or fillet at the top and bottom of a column, serving to divide the shaft of the column from its capital and base.

CINQUE-FOIL.

Cinque-foil.—A sinking or perforation, like a flower, of five points or leaves, as a quatre-foil is of four. The points are sometimes in a circle, and sometimes form the cusping of a head.

Civic Crown.—A garland of oak-leaves and acorns, given as honorary distinction among the Romans to such as had preserved the life of a fellow-citizen.

Clere-story, Clear-story.—When the middle of the nave of a church rises above the aisles and is pierced with windows, the upper story is thus called Sometimes these windows are very small, being mere quatre-foils, or spherical triangles. In large buildings, however, they are important objects, both for beauty and utility. The window of the clere-stories of Norman work, even in large churches, are of less importance than in the later styles. In Early English they became larger; and in the Decorated they are more important still, being lengthened as the triforium diminishes. In Perpendicular work the latter often disappears altogether, and in many later churches the clere-stories are close ranges of windows. The word *clere-story* is also used to denote a similar method of lighting other buildings besides churches. especially factories, depots, sheds, etc.

Cloister. An enclosed square, like the atrium of a Roman house, with a walk or ambulatory around sheltered by a roof, generally groined, and by tracery windows, which were more or less glazed

Close.—The precinct of a cathedral or abbey. Sometimes the walls are traceable, but now generally the boundary is only known by tradition

Bath Abbey.

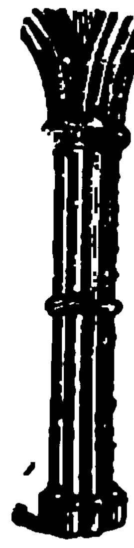
FLYING BUTTRESS AND CLERE-STORY.

A, buttress with pinnacle; B, flying buttress supporting clere-story; C, vaulted roof of aisle, D D, pier dividing nave from aisle; E, vaulted roof of nave.

Close String, or **Box String.** A method of finishing the outer edge of stairs, by building up a sort of curb string on which the balusters set, and the treads and risers stop against it.

Clustered.—In architecture, the coalition of several members which penetrate each other.

Clustered Column.—Several slender pillars attached to each other so as to form one. The term is used in Roman architecture to denote two or four columns which appear to intersect each other at the angle of a building to answer at each return.



CLUSTERED COLUMN.

Coat.—A thickness or covering of paint, plaster, or other work, done at one time. The first coat of plastering is called the scratch coat, the second coat (when there are three coats) is called the brown coat, and the last coat is variously known as the slipped coat, skim coat, or white coat. It varies in composition in different localities.

Coffer. A deep panel in a ceiling.

Coffer Dam. A frame used in the building of a bridge in deep water, similar to a caisson.

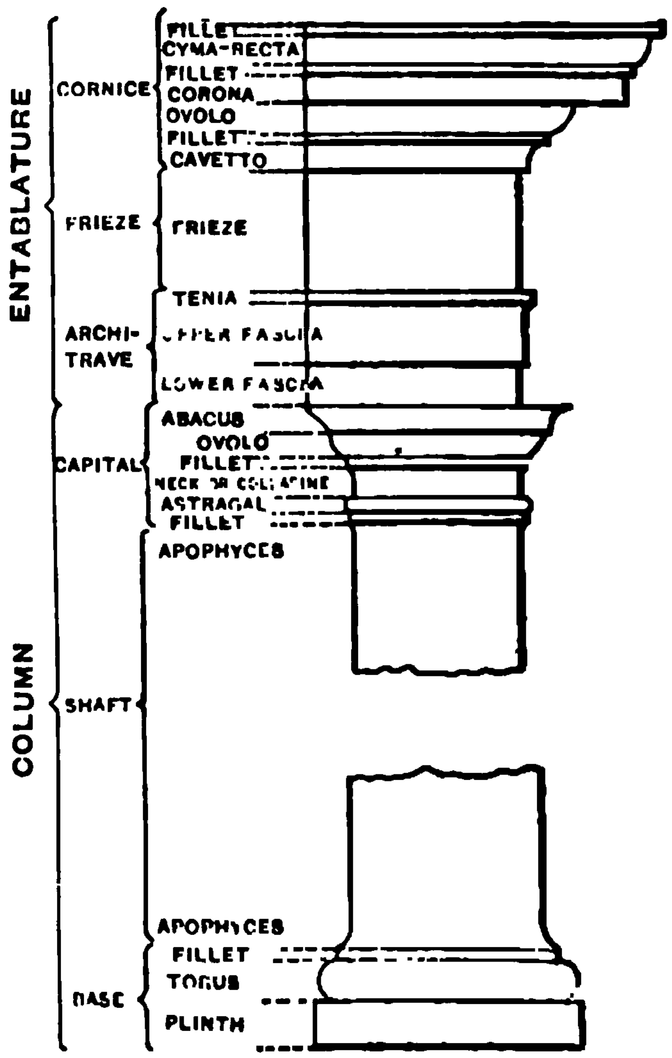
Collar Beam.—A beam above the lower ends of the rafters, and spiked to them.

Colonnade. A row of columns. The colonnade is termed, according to the number of columns which support the entablature: Tetrastyle, when there are four; hexastyle, when six; octostyle, when eight, etc. When in front of a building they are termed porticos; when surrounding a building, peristyle; and when double or more, polystyle.

Colosseum, or Coliseum. -The immense amphitheatre built at Rome by Flavius Vespasian, A.D. 72, after his return from his victories over the Jews. It would contain ninety thousand persons sitting, and twenty thousand more standing. The name is now employed to denote an unusually large audience building, generally of a temporary nature.

Colossus. The name of a brazen statue which was erected at the entrance of the harbor at Rhodes, one hundred and five feet in height. Vessels could sail between its legs.

Column. A round pillar. The parts are the base, on which it rests; its body, called the shaft; and the head, called the capital. The capital finishes with a horizontal table, called the abacus, and the base commonly stands on another, called the plinth. Columns may be either insulated or attached. They are said to be attached or engaged when they form part of a wall, projecting one-half or more, but not the whole, of their substance.



SECTION OF COLUMN AND ENTABLATURE. (Divided according to the Tuscan Order.)

Common. A line, angle, surface, etc., which belongs equally to several objects. Common centring is a centring without trusses, having a tie beam at bottom. Common joists are the beams in naked flooring to which the joists are fixed. Common rafters in a roof are those to which the laths are attached.

Composite Arch. Is the pointed or lancet arch.

Composite Order.—The most elaborate of the orders of classical architecture.

Concrete.—A mass composed of broken stone, sand, and hydraulic cement, which makes a sort of artificial stone, much used for foundations; a finer variety is sometimes used in blocks for building houses.

Conduit.—A long narrow passage between two walls or underground for secret communication between different apartments; also, a canal or pipe for the conveyance of water.

Confessional.—The seat where a priest or confessor sits to hear confessions.

Congé.—Another name for the echinus or quarter round.

Conservatory. A building for the protection and rearing of tender plants, often attached to a house as an apartment. Also, a public place of instruction, designed to preserve and perfect the knowledge of some branch of learning or the fine arts; as, a *conservatory* of music.

Consistory. The judicial hall of the College of Cardinals at Rome.

Consol, or Console. A bracket or truss, generally with scrolls or volutes at the two ends, of unequal size and contrasted, but connected by a flowing line from the back of the upper one to the inner convolving face of the lower.

Coping.—The capping or covering of a wall. This is of stone, weathered to throw off the wet. In Norman times, as far as can be judged from the little there is left, it was generally plain and flat, and projected over the wall with a throating to form a drip. Afterward it assumed a torus or bowtell at the top, and became deeper, and in the Decorated period there were generally several sets-off. The copings in the Perpendicular period assumed something of the wavy section of the buttress caps, and mitred round the sides of the embrasure, as well as the top and bottom.

CONSOLES.

Corbel.—The name, in mediæval architecture, for a piece of stone jutting out of a wall to carry any superincumbent weight. A piece of timber projecting in the same way was called a tassel or a bragger. Thus, the carved ornaments from which the vaulting shafts spring at Lincoln are corbels. Norman corbels are generally plain. In the Early English period they are sometimes elaborately carved. They sometimes end with a point, apparently growing into the wall, or forming a knot, and often are supported by angels and other figures. In the later periods the foliage or ornaments resemble those in the capitals. In modern architecture, a short piece of stone or wood projecting from a wall to form a support, generally ornamented.

Corbel Out.—To build out one or more courses of brick or stone from the face of a wall, to form a support for timbers.

Corbel Table. A projecting cornice or parapet, supported by a range of corbels a short distance apart, which carry a moulding, above which is a plain piece of projecting wall forming a parapet, and covered by a coping. Sometimes small arches are thrown across from corbel to corbel, to carry the projection.

Cornice. The projection at the top of a wall finished by a blocking-course, common in classic architecture. In Norman times, the wall finished with a corbel table, which carried a portion of plain projecting work, which was finished by a coping, and the whole formed a parapet. In Early English times the parapet was much the same, but the work was executed in a much better way, especially the small arches connecting the corbels. In the Decorated period the corbel table was nearly abandoned, and a large hollow, with one or two subordinate mouldings, substituted; this is sometimes filled with the ball flowers, and sometimes with running foliages. In the Perpendicular style, the parapet frequently

did not project beyond the wall-line below, the moulding then became a string (though often improperly called a cornice), and was ornamented by a quatre-foil, or small rosettes, set at equal intervals immediately under the battlements. In many French examples the moulded string is very bold, and enriched with foliage ornaments.

Corona. The brow of the cornice which projects over the bed mouldings to throw off the water.

Corridor. A long gallery or passage in a mansion connecting various apartments and running round a quadrangle. Any long passage-way in a building.

Countersink. To make a cavity for the reception of a plate of iron, or the head of a screw or bolt, so that it shall not project beyond the face of the work.

Coupled Columns. Columns arranged in pairs.

Course. A continued layer of bricks or stones in buildings; the term is also applicable to slates, shingles, etc.

Court. An open area behind a house, or in the centre of a building and the wings. Courts admit of the most elegant ornamentations, such as arcades, etc.

Cove Coving.—The moulding called the cavetto, or the scotia inverted, on a large scale, and not as a mere moulding in the composition of a cornice, is called a cove or a coving.

Cove-bracketing.—The wooden skeleton mould or framing of a cove, applied chiefly to the bracketing of a cove ceiling.

Cove Ceiling. A ceiling springing from the walls with a curve.

Coved and Flat Ceiling. A ceiling in which the section is the quadrant of a circle, rising from the walls and intersecting in a flat surface.

Cradling. Timber work for sustaining the lath and plaster of vaulted ceilings.

Cresting. An ornamental finish in the wall or ridge of a building, which is common on the Continent of Europe. An example occurs at Exeter Cathedral, the ridge of which is ornamented with a range of small fleurs-de-lis in lead.

Crocket. An ornament running up the sides of gablets, hood-moulds, pinnacles, spires, generally a winding stem like a creeping plant, with flowers or leaves projecting at intervals, and terminating in a finial.

CROCKET.

Cross. This religious symbol is almost always placed on the ends of gables, the summit of spires, and other conspicuous parts of old churches. In early times it was generally very plain, or [illegible] a simple cross in a circle. Sometimes they take the form of a [illegible] cross, crosslet, or a cross in a square. In the [illegible] styles they became richly floriated, and assumed an endless variety of forms. Of memorial crosses the finest examples are the Eleanor crosses, erected by Edward I. Of these a few yet remain, one of which has [illegible] at Charing Cross. Preaching crosses were often set up [illegible] as stations for preaching; the most noted is that in front of St. Paul's, London. The finest remaining sepulchral crosses are the old elaborately carved examples found in Ireland.

Cross-aisle. An old name for a transept.

Cross-springer.—The transverse ribs of a vault.

Cross-vaulting. A common name given to groins and cylindrical vaults.

Crown. In architecture, the uppermost member of the cornice; called also Corona and Larmier.

Crypt. A vaulted apartment of greater or less size, usually under the choir

Cupola.—A small room, either circular or polygonal, standing on the top of a dome. By some it is called a Lantern.

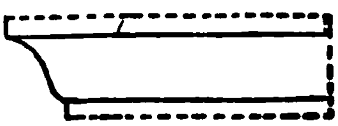
Curb Roof, or **Mansard Roof.**—A roof formed of four contiguous planes, each two having an external inclination.

Curtail Step.—The first step in a stair, which is generally finished in the form of a scroll.

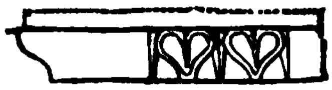
Cusp.—The point where the foliations of tracery intersect. The earliest example in England of a plain cusp is probably that at Pythagoras School, at Cambridge; of an ornamental cusp, at Ely Cathedral, where a small roll, with a rosette at the end, is formed at the termination of a cusp. In the later styles the terminations of the cusps were more richly decorated; they also sometimes terminate not only in leaves or foliages, but in rosettes, heads, and other fanciful ornaments.

Cyclostyle.—A structure composed of a circular range of columns without a core is cyclostylar; with a core, the range would be a peristyle. This is the species of edifice called by Vitruvius monopteral.

Cyma.—The name of a moulding of very frequent use. It is a simple, waved line, concave at one end and convex at the other, like an Italic *f*. When the concave part is uppermost it is called a cyma recta; but if the convexity appear above, and the concavity below, it is then a cyma reversa.



CYMA RECTA.



CYMA REVERSA.

Cymatium.—When the crowning moulding of an entablature is of the cyma form, it is termed the Cymatium.

Cyrtostyle.—A circular projecting portico. Such are those of the transept entrances to St. Paul's Cathedral, London.

Dado, or **Die.**—The vertical face of an insulated pedestal between the base and cornice, or surbase. It is extended also to the similar part of all stereobates which are arranged like pedestals in Roman and Italian architecture.

Dais.—A part of the floor at the end of a mediæval hall, raised a step above the rest of the floor. On this the lord of the mansion dined with his friends at the great table, apart from the retainers and servants. In mediæval halls there was generally a deep recessed bay window at one or at each end of the dais, supposed to be for retirement, or greater privacy than the open hall could afford. In France the word is understood as a canopy or hanging over a seat; probably the name was given from the fact that the seats of great men were then surmounted by such an ornament.

Darby.—A flat tool used by plasterers in working, especially on ceilings. It is generally about seven inches wide and forty-two inches long, with two handles on the back.

Decastyle.—A portico of ten columns in front.

Decorated Style.—The second stage of the Pointed or Gothic style of architecture, considered the most complete and perfect development of Gothic architecture, the best examples of which are found in England.

Demi-metope.—The half of a metope, which is found at the retiring or projecting angles of a Doric frieze.

Dentil.—The cogged or toothed member, common in the bed-mould of a Corinthian entablature, is said to be dentilled, and each cog or tooth is called a dentil.

Depressed Arches, or **Drop Arches.**—Those of less pitch than the equilateral.

Design.—The plans, elevations, sections, and whatever other drawings may be necessary for an edifice, exhibit the design, the term plan having a restricted application to a technical portion of the design.

Detail.—As used by architects, detail means the smaller parts into which a

composition may be divided. It is applied generally to mouldings and other enrichments, and again to their minutiæ.

Diameter.—The line in a circle passing through its centre, or thickest part, which gives the measure proportioning the intercolumniation in some of the orders.

Diameters. The diameters of the lower and upper ends of the shaft of a column are called its inferior and superior diameters, respectively; the former is the greatest, the latter the least diameter of the shaft.

Diaper.—A method of decorating a wall, panel, stained glass, or any plain surface, by covering it with a continuous design of flowers, rosettes, etc., either in squares or lozenges, or some geometrical form resembling the pattern of a diapered table-cloth, from which, in fact, the name is supposed by some to have been derived.

Diastyle.—A spacious intercolumniation, to which three diameters are assigned.

Dipteros. A double-winged temple. The Greeks are said to have constructed temples with two ranges of columns all around, which were called dipteroi. A portico projecting two columns and their interspaces is of dipteral or pseudodipteral arrangement.

Discharging Arch.—An arch over the opening of a door or window, to discharge or relieve the superincumbent weight from pressing on the lintel.

Distemper.—Term applied to painting with colors mixed with size or other glutinous substance. All the cartoons of the ancients, previous to the year 1410, are said to be done in distemper.

Distyle. A portico of two columns. This is not generally applied to the mere porch with two columns, but to describe a portico with two columns *in antis*.

Ditriglyph.—An intercolumniation in the Doric order, of two triglyphs.

Dodecastyle.—A portico of twelve columns in front. The lower one of the west front of St. Paul's Cathedral, London, is of twelve columns, but they are coupled, making the arrangement pseudo dodecastyle. The Chamber of Deputies in Paris has a true dodecastyle.

Dog-tooth. A favorite enrichment used from the latter part of the Norman period to the early part of the Decorated. It is in the form of a four-leaved flower, the centre of which projects, and probably was named from its resemblance to the dog toothed violet.

Dome. A cupola or inverted cup on a building. The application of this term to its generally received purpose is from the Italian custom of calling an archiepiscopal church, by way of eminence, Il Duomo, the temple; for to one of that rank, the Cathedral of Florence, the cupola was first applied in modern practice. The Italians themselves never call a cupola a dome; it is on this side of the Alps the application has arisen, from the circumstance, it would appear, that the Italians use the term with reference to those structures whose most distinguishing feature is the cupola, tholus, or (as we now call it) dome.

Domestic Architecture. That branch which relates to private buildings.

Donjon. The principal tower of a castle, generally containing the prison.

Door Frame. The surrounding case into and out of which the door shuts and opens. It consists of two upright pieces, called jambs, and a head, generally fixed together by mortices and tenons, and wrought, rebated, and beaded.

Doric Order. The oldest of the three orders of Grecian architecture.

Dormer Window. A window belonging to a room in a roof, which consequently projects from it with a valley gutter on each side. They are said not to be earlier than the fourteenth century. In Germany there are often several rows

of dormers, one above the other. In Italian Gothic they are very rare ; in fact, the former have an unusually steep roof, while in the latter country, where the Italian tile is used, the roofs are rather flat.

Dormitory.—A room, suite of rooms, or building used to sleep in. The name was first applied to the place where the monks slept at night. It was sometimes one long room like a barrack, and sometimes divided into a succession of small chambers or cells. The dormitory was generally on the first floor, and connected with the church, so that it was not necessary to go out-of-doors to attend the nocturnal services. In the large houses of the Perpendicular period, and also in some of the Elizabethan, the entire upper story in the roof formed one large apartment, said to have been a place for exercise in wet weather, and also for a dormitory for the retainers of the household, or those of visitors.

Double Vault.—Formed by a duplicate wall ; wine cellars are sometimes so formed.

Dovetailing.—In carpentry and joinery, the method of fastening boards or other timbers together, by letting one piece into another in the form of the expanded tail of a dove.

Dowel.—1. A pin let into two pieces of wood or stone, where they are joined together. 2. A piece of wood driven into a wall so that other pieces may be nailed to it. This is also called plugging.

Draw-bridge.—A bridge made to draw up or let down, much used in fortified places. In navigable rivers, the arch over the deepest channel is made to draw or revolve, in order to let the masts of ships pass through.

Drawing-room.—A room appropriated for the reception of company ; a room to which company withdraws from the dining-room.

Dresser.—A cupboard or set of shelves to receive dishes and cooking utensils.

Dressing.—Is the operation of squaring and smoothing stones for building ; also applied to smoothing lumber.

Dressing-room.—An apartment appropriated for dressing the person.

Drip.—A name given to the member of a cornice which has a projection beyond the other parts for throwing off water by small portions, drop by drop. It is also called Larmier.

Drip-stone.—The label moulding which serves on a canopy for an opening, and to throw off the rain. It is also called Weather Moulding.

Drop-scene.—A curtain suspended by pulleys, which descends or drops in front of the stage in a theatre.

Drum.—The upright part of a cupola over a dome ; also, the solid part or vase of the Corinthian and Composite capitals.

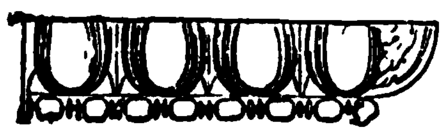
Dry-rot.—A rapid decay of timber, by which its substance is converted into a dry powder, which issues from minute cavities resembling the borings of worms.

Dungeon.—The prison in a castle keep, so called because the Norman name for the latter is donjon, and the dungeons, or prisons, are generally in its lowest story.

Dwarf Wall.—The walls enclosing courts above which are railings of iron ; low walls, in general, receive this name.

Eaves.—In slating and shingling, the margin or lower part of the slating hanging over the wall, to throw the water off from the masonry or brickwork.

Echinus.—A moulding of eccentric curve, generally cut (when it is carved) into the forms of eggs and anchors alternating, whence the moulding is called by the name of the more conspicuous. It is the same as Ovolo.



ECHINUS.

Edifice.—Is synonymous with the terms building, fabric, erection, but is more strictly applicable to architecture distinguished for size, dignity, and grandeur.

Efflorescence.—In architecture, the formation of a whitish loose powder, or crust, on the surface of stone or brick walls.

Egyptian Architecture.—The earliest civilization and cultivation of the arts was in Upper Egypt. The most remarkable and most ancient monuments of the Egyptians, with the exception of the pyramids, are nearly all included in Upper Egypt. The buildings of Egypt are characterized by solidity and massiveness of construction, originality of conception, and boldness of form. The walls, the pillars, and the most sacred places of their religious buildings were ornamented with hieroglyphics and symbolical figures, while the ceilings of the porticos exhibited zodiacs and celestial planispheres. The temples of Egypt were generally without roofs, and, consequently, the interior colonnades had no pediments, supporting merely an entablature, composed of only architrave, frieze, and cornice, formed of immense blocks united without cement and ornamented with hieroglyphics.

Element. The outline of the design of a Decorated window, on which the centres for the tracery are formed. These centres will all be found to fall on points which, in some way or other, will be equimultiples of parts of the openings. To draw tracery well, or understand even the principles of its composition, much attention should be given to the study of the element.

Elevation. The front façade, as the French term it, of a structure; a geometrical drawing of the external upright parts of a building.

Embattlement. An indented parapet; battlement.

Emblazon. To adorn with figures of heraldry, or ensigns armorial.

Embossing. Sculpture in rilievo, the figures standing partly out from the plane.

Embrasure. The opening in a battlement between the two raised solid portions or merlons, sometimes called a crenelle.

Encaustic. Pertaining to the art of burning in colors, applied to painting on glass, porcelain, or tiles, where colors are fixed by heat; hence, encaustic tiles, brick, etc.

Engaged Columns. Are those attached to, or built into walls or piers, a portion being concealed.

Enrichment. The addition of ornament, carving, etc., to plain work; decoration; embellishment.

Ensemble. Means the whole work or composition considered together, and not in parts.

Entablature. The assemblage of parts supported by the column. It consists of three parts: the architrave, frieze, and cornice.

Entail. In Gothic architecture, delicate carving.

Entasis. The swelling of a column, etc. In mediæval architecture, some spires, particularly those called "broach spires," have a slight swelling in the sides, but no more than to make them look straight; for, from a particular [illegible]," that which is quite straight, when viewed at a height, looks hollow.

Entry. A hall without stairs or vestibule.

Epistyle. This term may with propriety be applied to the whole entablature, with which it is synonymous; but it is restricted in use to the architrave, or lowest member of the entablature.

Escutcheon. (Her.) The field or ground on which a coat-of-arms is represented. (Arch.) The shields used on tombs, in the spandrels of doors, or in

string-courses ; also, the ornamented plates from the centre of which door rings, knockers, etc., are suspended, or which protect the wood of the key-hole from the wear of the key. In mediæval times these were often worked in a very beautiful manner.

Etching.—A mode of engraving on glass or metal (generally copper) by means of lines, eaten in or corroded by means of some strong acid.

Eustyle.—A species of intercolumniation to which a proportion of two diameters and a quarter is assigned. This term, together with the others of similar import—pycnostyle, systyle, diastyle, and aræostyle - referring to the distances of columns from one another in composition, is from Vitruvius, who assigns to each the space it is to express. It will be seen, however, by reference to them individually, that the words themselves, though perhaps sufficiently applicable, convey no idea of an exactly defined space, and, by reference to the columnar structures of the ancients, that no attention was paid by them to such limitations. It follows, then, that the proportions assigned to each are purely conventional, and may or may not be attended to without vitiating the power of applying the terms. Eustyle means the best or most beautiful arrangement ; but, as the effect of a columnar composition depends on many things besides the diameter of the columns, the same proportioned intercolumniation would look well or ill according to those other circumstances, so that the limitation of Eustyle to two diameters and a quarter is absurd.

Extrados.—The exterior or convex curve forming the upper line of the arch stones ; the term is opposed to the intrados, or concave side.

Eye of a Dome.—The aperture at its summit.

Eye of a Volute.—The circle in its centre.

Facade, or **Face.**—The whole exterior side of a building that can be seen at one view ; strictly speaking, the principal front.

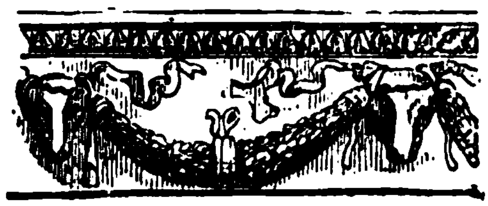
Face Mould.—The pattern for marking the plank or board out of which ornamental hand-railings for stairs and other works are cut.

Fan Tracery.—The very complicated mode of roofing used in the Perpendicular style, in which the vault is covered by ribs and veins of tracery.

Fascia.—A flat, broad member in the entablature of columns or other parts of buildings, but of small projection. The architraves in some of the orders are composed of three bands, or fasciæ : the Tuscan and the Doric ought to have only one. Ornamental projections from the walls of brick buildings over any of the windows, except the uppermost, are called Fasciæ.

Fenestral.—A frame, or "chassis," on which oiled paper or thin cloth was strained to keep out wind and rain when the windows were not glazed.

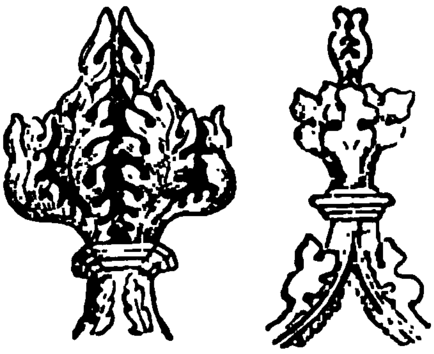
Festoon.—An ornament of carved work, representing a wreath or garland of flowers or leaves, or both, interwoven with each other. It is thickest in the middle, and small at each extremity, where it is tied, a part often hanging down below the knot.



FESTOON.

Fillet.—A narrow vertical band or listel, of frequent use in congeries of mouldings, to separate and combine them, and also to give breadth and firmness to the upper edge of a crowning cyma or cavetto, as in an external cornice. The narrow slips or breadth between the flutes of Corinthian and Ionic columns are also called fillets. In mediæval work the fillet is a small, flat, projecting square, chiefly used to separate hollows and rounds, and often found in the outer parts of shafts and boutels. In this situation the centre fillet has been termed a keel, and the two side ones, wings ; but, apparently, this is not an ancient usage.

Finial.—The flower, or bunch of flowers, with which a spire, pinnacle, gablet, canopy, etc., generally terminates. Where there are crockets, the finial generally bears as close a resemblance as possible to them in point of design. They are found in early work where there are no crockets. The simplest form more resembles a bud about to burst than an open flower. They soon became more elaborate, as at Lincoln, and still more, as at Westminster and the Hôtel Cluny at Paris. Many perpendicular finials are like four crockets bound together. Almost every known example of a finial has a sort of necking separating it from the parts below.



FINIALS.

Fish-joint. A splice where the pieces are joined butt end to end, and are connected by pieces of wood or iron placed on each side and firmly bolted to the timbers, or pieces joined. (See Chapter XXIX.)

Flags. Flat stones, from 1 to 3 inches thick, for floors.

Flamboyant. A name applied to the Third Pointed style in France, which seems to have been developed from the Second, as the English Perpendicular was from the Decorated. The great characteristic is, that the element of the tracery flows upward in long wavy divisions like flames of fire. In most cases, also, every division has only one cusp on each side, however long the division may be. The mouldings seem to be as much inferior to those of the preceding period as the Perpendicular mouldings were to the Early English, a fact which seems to show that the decadence of Gothic architecture was not confined to one country.

Flange. A projecting edge, rib, or rim. Flanges are often cast on the top or bottom of iron columns, to fasten them to those above or below; the top and bottom of I beams and channels are called the flange.

Flashings. Pieces of lead, tin, or copper, let into the joints of a wall so as to lap over gutters or other pieces; also, pieces worked in the slates or shingles around dormers, chimneys, and any rising part, to prevent leaking.

Flatting. Painting finished without leaving a gloss on the surface.

Flèche. A general term in French architecture for a spire, but more particularly used for the small, slender erection rising from the intersection of the nave and transepts in cathedrals and large churches, and carrying the sanctus bell.

Flight. A run of steps or stairs from one landing to another.

Floating. The equal spreading of plaster or stucco on the surface of walls, by means of a board called a float; as a rule, only rough plastering is floated.

Floriated. Having florid ornaments, as in Gothic pillars.

Fleur-de-lis. The royal insignia of France, much used in decoration.

Flue. The space or passage in a chimney through which the smoke ascends. Each passage is called a flue, while all together make the chimney.

Flush. The continued surface, in the same plane, of two contiguous masses.

Flute. A concave channel. Columns whose shafts are channelled are said to be fluted, and the flutes are collectively called Flutings.

Flying Buttress. An arched buttress used when extra strength was required [illegible] part of the wall of the nave, etc., to resist the outward thrust of a [illegible] The flying buttress generally rests on the wall and buttress of [illegible]

Foils. [illegible] small arcs in the tracery of Gothic windows, panels, etc.

Foliage. An ornamental distribution of leaves on various parts of buildings.

Foliation. The use of small arcs or foils in forming tracery

Font. The vessel used in the rite of baptism. The earliest extant is supposed

to be that in which Constantine is said to have been baptized; this is a porphyry labrum from a Roman bath. Those in the baptisteries in Italy are all large, and were intended for immersion; as time went on, they seem to have become smaller. Fonts are sometimes mere plain hollow cylinders, generally a little smaller below than above; others are massive squares, supported on a thick stem, round which sometimes there are smaller shafts. In the Early English this form is still pursued, and the shafts are detached; sometimes, however, they are hexagonal and octagonal, and in this and the later styles assume the form of a vessel on a stem. Norman fonts have frequently curious carvings on them, approaching the grotesque; in later times the foliages, etc., partook absolutely of the character of those used in other architectural details of their respective periods. The font in European churches is usually placed close to a pillar near the entrance, generally that nearest but one to the tower in the south arcade; or, in large buildings, in the middle of the nave, opposite the entrance porch, and sometimes in a separate building. In Protestant churches in this country, the font is generally placed inside the communion rail, or on the steps of the chancel.

Footings.—The spreading courses at the base or foundation of a wall. When a layer of different material from that of the wall (as a bed of concrete) is used, it is called the Footing.

Foundation.—That part of a building or wall which is below the surface of the ground.

Foxtail Wedging.—Is a peculiar mode of mortising, in which the end of the tenon is notched beyond the mortise, and is split and a wedge inserted, which, being forcibly driven in, enlarges the tenon and renders the joint firm and immovable.

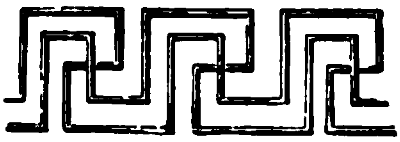
Frame.—The name given to the wood-work of windows, doors, etc.; and in carpentry, to the timber works supporting floors, roofs, etc.

Framing.—The rough timber work of a house, including the flooring, roofing, partitioning, ceiling, and beams thereof.

Freestone.—Stone which can be used for mouldings, tracery, and other work required to be executed with the chisel. The oölitic and sandstones are those generally included by this term.

Fresco.—The method of painting on a wall while the plastering is wet. The color penetrates through the material, which, therefore, will bear rubbing or cleaning to almost any extent. The transparency, the chiaro-oscuro, and lucidity, as well as force, which can be obtained by this method, cannot be conceived unless the frescos of Fra Angelico or Raffaelle are studied. The word, however, is often applied improperly to painting on the surface in distemper or body color, mixed with size or white of egg, which gives an opaque effect.

Fret.—An ornament consisting of small fillets intersecting each other at right angles.



FRET.

Frieze.—That portion of an entablature between the cornice above and architrave below. It derives its name from being the recipient of the sculptured enrichments either of foliage or figures which may be relevant to the object of the sculpture. The frieze is also called the Zoöphorus.

Frigidarium.—An apartment in the Roman bath, supplied with cold water.

Furniture.—A name given to the metal trimmings of doors, windows, and other similar parts of a house. In this country the word "hardware" is more generally used to denote the same thing.

Furrings.—Flat pieces of timber used to bring an irregular framing to an even surface.

Gable. When a roof is not hipped or returned on itself at the ends, its ends are stopped by carrying up the walls under them in the triangular form of the roof itself. This is called the gable, or, in the case of the ornamental and ornamented gable, the pediment. Of necessity, gables follow the angles of the slope of the roof, and differ in the various styles. In Norman work they are generally about half pitch; in Early English, seldom less than equilateral, and often more. In Decorated work they become lower, and still more so in the Perpendicular style. In all important buildings they are finished with copings or parapets. In the later Gothic styles gables are often surmounted with battlements, or enriched with crockets; they are also often panelled or perforated, sometimes very richly. The gables in ecclesiastical buildings are mostly terminated with a cross; in others, by a finial or pinnacle. In later times the parapets or copings were broken into a sort of steps, called corbie steps. In buildings of less pretension the tiles or other roof covering passed over the front of the wall, which then, of course, had no coping. In this case, the outer pair of rafters were concealed by moulded or carved verge boards.

Gable Window. A term sometimes applied to the large window under a gable, but more properly to the windows in the gable itself.

Gabled Towers.—Those which are finished with gables instead of parapets. Many of the German Romanesque towers are gabled.

Gablets. Triangular terminations to buttresses, much in use in the Early English and Decorated periods, after which the buttresses generally terminate in pinnacles. The Early English gablets are generally plain, and very sharp in pitch. In the Decorated period they are often enriched with panelling and crockets. They are sometimes finished with small crosses, but oftener with finials.

Gain. A bevelled shoulder on the end of a mortised brace, for the purpose of giving additional resistance to the shoulder.

Gallery. Any long passage looking down into another part of a building, or into the court outside. In like manner, any stage erected to carry a rood or an organ, or to receive spectators, was latterly called a gallery, though originally a loft. In later times the name was given to any very long rooms, particularly those intended for purposes of state, or for the exhibition of pictures.

Gambrel Roof.—A roof with two pitches, similar to a mansard or curb roof.

Gargoyle, or Gurgoyle.—The carved termination to a spout which conveyed away the water from the gutters; supposed to be called so from the gurgling noise made by the water passing through it. Gargoyles are mostly grotesque figures.

GARGOYLE.

Gate-house. A building forming the entrance to a town, the door of an abbey, or the enceinte of a castle or other important edifice. They generally had the archway protected by a gate, and also a portcullis, over which were battlemented parapets with loopholes and machicolations for throwing down darts, melted lead, or hot sand on the besiegers. Gate-houses always had a lodge, with apartments for the porter and guard-rooms for the soldiers, and generally rooms over for the officers, and often places for prisoners beneath. The name is now generally only applied to the gate-keeper's lodge on large estates.

Gauge. 1. To mix plaster of Paris with common plaster to make it set quick, called gauged mortar. 2. A tool used by carpenters, to strike a line parallel to the edge of a board.

—A large timber or iron beam, either single or built up, used to sup- or walls over an opening.
—A vertical channel in a frieze.
Style. The name of Gothic was given to the various Medieval styles d in the sixteenth century when a great classic revival was going on, thing not classic was considered barbarian, or Gothic. The term was nally intended as one of stigma, and, although it conveys a false idea of ter of the Medieval styles, it has long been used to distinguish them Grecian and Roman. The true principle of Gothic architecture is the ivision, relation and subordination of the different parts, distinct and ty with each other, and while this principle was adhered to, Gothic re may be said to have retained its vitality.
.—A word derived from the French, signifying a large barn or granary. vere usually long buildings with high wooden roofs, sometimes divided r columns into a sort of nave and aisles, with walls strongly buttressed d the term was applied not only to the barns, but to the whole of the which formed the detached farms belonging to the monasteries; in s there was a chapel either included among these or standing apart as a dific e.
e. A framework of beams laid longitudinally and crossed by similar tched upon them, used to sustain walls to prevent irregular setting.
—The iron work forming the enclosure screen to a chapel, or the pro- iling to a tomb or shrine, more commonly found in France than in They are of wrought iron, ornamented by the swage and punch, and her either by rivets or clips. In modern times grilles are used exten- protecting the lower windows in city houses, also the glass opening in oors.
—By some described as the line of intersection of two vaults where they other, which others call the groin point; by others the curved section l of such vault ng is called a groin, and by others the whole system of s so named.
Arch. The cross-rib in the later styles g, passing at right angles from wall to dividing the vault into bays or travees.
Ceiling.—A ceiling to a building com- ak ribs, the spandrels of which are filled arrow, thin slips of wood. There are England; one at the Early English Warmington, and one at Winchester, exactly resembling those of stone.
Centring. In groining without ribs, surface is supported by centring during on of the vaulting. In ribbed work the only are supported by timber ribs dur- ogress of the work, any light stuff being used while filling in the span-

GROINED VAULTING.

Point. The name given by workmen to the arris or line of intersec- e vault with another where there are no ribs.
Rib. The rib which conceals the groin point or joints, where the span- meet.
d Vaulting.—The system of covering a building with stone vaults es and intersect each other, as opposed to the barrel vaulting, or series placed side by side. The earliest groins are plain, without any ribs,

except occasionally a sort of wide band from wall to wall, to strengthen the construction. In later Norman times ribs were added on the line of intersection of the spandrels, crossing each other, and having a boss as a key common to both; these ribs the French authors call *nerfs en ogive*. Their introduction, however, caused an entire change in the system of vaulting; instead of arches of uniform thickness and great weight, these ribs were first put up as the main construction, and spandrels of the lightest and thinnest possible material placed upon them, the haunches only being loaded sufficiently to counterbalance the pressure from the crown. Shortly after, half-ribs against the walls (formerets) were introduced to carry the spandrels without cutting into the walling, and to add to the appearance. The work was now not treated as continued vaulting, but as divided into bays, and it was formed by keeping up the ogive, or intersecting ribs and their bosses; a sort of construction having some affinity to the dome was formed, which added much to the strength of the groining. Of course, the top of the soffit or ridge of the vault was not horizontal, but rose from the level of the top of the formeret-rib to the boss and fell again; but this could not be perceived from below. As this system of construction got more into use, and as the vaults were required to be of greater span and of higher pitch, the spandrels became larger, and required more support. To give this, another set of ribs was introduced, passing from the springers of the ogive ribs, and going to about half-way between these and the ogive, and meeting on the ridge of the vault; these intermediate ribs are called by the French *tiercerons*, and began to come into use in the transition from Early English to Decorated. About the same period a system of vaulting came into use called *hexpartite*, from the fact that every bay is divided into six compartments instead of four. It was invented to cover the naves of churches of unusual width. The filling of the spandrels in this style is very peculiar, and, where the different compartments meet at the ridge, some pieces of harder stone have been used, which give rather a pleasing effect. The arches against the wall, being of smaller span than the main arches, cause the centre springers to be perpendicular and parallel for some height, and the spandrels themselves are very hollow. As styles progressed, and the desire for greater richness increased, another series of ribs, called *liernes*, was introduced; these passed crossways from the *ogives* to the *tiercerons*, and thence to the *doubleaux*, dividing the spandrels nearly horizontally. These various systems increased in the Perpendicular period, so that the vaults were quite a net-work of ribs, and led at last to the Tudor, or, as it is called by many, fan-tracery vaulting. In this system the ribs are no part of the real construction, but are merely carved upon the voussoirs, which form the actual vaulting. Fan Tracery is so called because the ribs radiate from the springers, and spread out like the sticks of a fan. These later methods are not strictly groins, for the pendentives are not square on plan, but circular, and there is, therefore, no arris intersection or groin point.

Groins, Welsh, or Underpitch. When the main longitudinal vault of any groining is higher than the cross or transverse vaults which run from the windows, the system of vaulting is called underpitch groining, or, as termed by the workmen, Welsh groining. A very fine example is at St. George's Chapel, Windsor, England.

Groove. In joinery, a term used to signify a sunk channel whose section is rectangular. It is usually employed on the edge of a moulding, stile, or rail, etc., into which a tongue corresponding to its section, and in the substance of the wood to which it is joined, is inserted.

Grotesque. A singular and fantastic style of ornament found in ancient buildings.

Grotto. An artificial cavern.

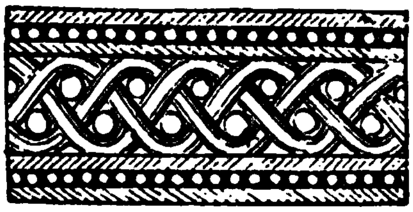
Ground Floor.—The floor of a building on a level, or nearly so, with the ground.

Ground Joist.—Joist that is blocked up from the ground.

Grounds.—Pieces of wood embedded in the plastering of walls to which skirting and other joiner's work is attached. They are also used to stop the plastering around door and window openings.

Grouped Columns.—Three, four, or more columns put together on the same pedestal. When two are placed together, they are said to be coupled.

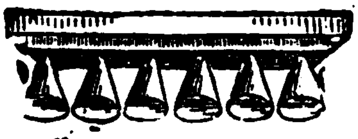
Grout.—Mortar made so thin by the addition of water that it will run into all the joints and cavities of the mason-work, and fill it up solid.



GUILLOCHE.

Guilloche, or Guillochos.—An interlaced ornament like net-work, used most frequently to enrich the torus.

Guttæ.—The small cylindrical drops used to enrich the mutules and regulæ of the Doric entablature are so called.



GUTTÆ.

Gutter.—The channel for carrying off rain-water. The mediæval gutters differed little from others, except that they are often hollows sunk in the top of stone cornices, in which case they are generally called channels in English, and *cheneaux* in French.

Gymnasium.—A building classed in the first rank by the Greeks; it was in them they instructed the youth in all the arts of peace and war; a building for athletic exercises.

Hall.—1. The principal apartment in the large dwellings of the Middle Ages, used for the purposes of receptions, feasts. etc. In the Norman castle the hall was generally in the keep above the ground floor, where the retainers lived, the basement being devoted to stores and dungeons for confining prisoners. Later halls—indeed, some Norman halls (not in castles)—are generally on the ground floor, as at Westminster, approached by a porch either at the end, as in this last example, or at the side, as at Guildhall, London, having at one end a raised dais or estrade. The roofs are generally open and more or less ornamented. In the middle of these was an opening to let out the smoke, though in later times the halls have large chimney-places with funnels or chimney-shafts for this purpose. At this period there were usually two deeply recessed bay windows at each end of the dais, and doors leading into the withdrawing-rooms, or the ladies' apartments; they are also generally wainscoted with oak, in small panels, to the height of five or six feet, the panels often being enriched. Westminster Hall was originally divided into three parts, like a nave and side aisles, as are some on the Continent of Europe. 2. A room or passage-way at the entrance of a house, or suite of chambers. 3. A place of public assembly, as a town-hall, a music-hall.

Halving.—The junction of two pieces of timber, by letting one into the other.

Hammer Beam.—A beam in a Gothic roof, not extending to the opposite side; a beam at the foot of a rafter.

Hanging Buttress.—A buttress not rising from the ground, but supported on a corbel, applied chiefly as a decoration and used only in the Decorated and Perpendicular style.

Hanging Stile.—Of a door, is that to which the hinges are fixed.

Hangings.—Tapestry; originally invented to hide the coarseness of the

walls of a chamber. Different materials were employed for this purpose, some of them exceedingly costly and beautifully worked in figures, gold and silk.

Hatching.—Drawing parallel lines close together for the purpose of indicating a section of anything. The lines are generally drawn at an angle of 45° with a horizontal.

Haunches.—The sides of an arch, about half-way from the springing to the crown.

Headers.—In masonry, are stones or bricks extending over the thickness of a wall. In carpentry, the large beam into which the common joists are framed in framing openings for stairs, chimneys, etc.

Heading Courses.—Courses of a wall in which the stone or brick are all headers.

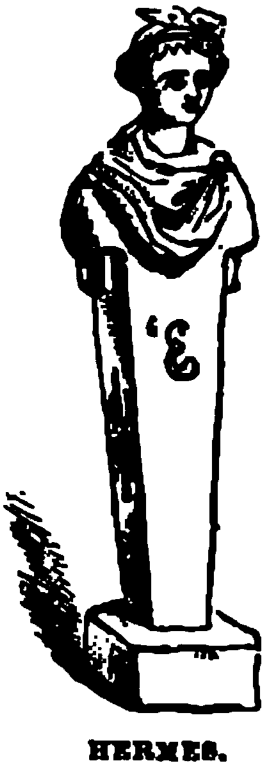
Head-way.—Clear space or height under an arch, or over a stairway, and the like.

Heel.—Of a rafter, the end or foot that rests upon the wall plate.

Height.—Of an arch, a line drawn from the middle of the chord to the intrados.

Helix. A small volute or twist like a stalk, representing the twisted tops of the acanthus, placed under the abacus of the Corinthian capital.

Hermes.—A rough quadrangular stone or pillar, having a head, usually of Hermes or Mercury, sculptured on the top, without arms or body, placed by the Greeks in front of buildings.



HERMES.

Herring-bone Work.—Bricks, tile, or other materials arranged diagonally in building.

Hexastyle.—A portico of six columns in front is of this description.

High Altar. The principal altar in a cathedral or church. Where there is a second, it is generally at the end of the choir or chancel, not in the lady chapel.

Hip-knob. The finial on the hip of a roof, or between the barge boards of a gable.

Hip-roof. A roof which rises by equally inclined planes from all four sides of the building.

Hippodrome.—A place appropriated by the ancients for equestrian exercises.

Hips. Those pieces of timber placed in an inclined position at the corners or angles of a hip-roof.

Hood-mould.—A word used to signify the drip-stone for label over a window or door opening, whether inside or out.

Hôtel de Ville.—The town-hall, or guild-hall, in France, Germany, and Northern Italy. The building, in general, serves for the administration of justice, the receipt of town dues, the regulation of markets, the residence of magistrates, barracks for police, prisons, and all other fiscal purposes. As may be imagined, they differ very much in different towns, but they have almost invariably attached to them, or closely adjacent, a large clock-tower containing one or more bells, for calling the people together on special occasions.

Hôtel Dieu. The name for a hospital in mediæval times. In England there are but few remains of these buildings, one of which is at Dover; in France there are many. The most celebrated is the one at Angers, described by Parker. They do not seem to differ much in arrangement of plan from those in modern days, the accommodation for the chaplain, medicine, nurses, stores, etc., being much the same in all ages, except that in some of the earlier, instead of the sick

being placed in long wards like galleries, as is now done, they occupied large buildings, with naves and side aisles, like churches.

Housing.—The space taken out of one solid to admit the insertion of another. The base on a stair is generally housed into the treads and risers; a niche for a statue.

Hypæthros.—A temple open to the air, or uncovered. The term may be the more easily understood by supposing the roof removed from over the nave of a church in which columns or piers go up from the floor to the ceiling, leaving the aisles still covered.

Hypogea.—Constructions under the surface of the earth, or in the sides of a hill or mountain.

Ichnography.—A horizontal section of a building or other object, showing its true dimensions according to a geometric scale; a ground plan.

Impluvium.—The central part of an ancient Roman court, which was uncovered.

Impost.—A term in classic architecture for the horizontal mouldings of piers or pilasters, from the top of which spring the archivolts or mouldings which go round the arch.

In Antis.—When there are two columns between the antæ of the lateral walls and the cella.

Incise.—To cut in; to carve; to engrave.

Indented.—Toothed together.

Inlaying.—Inserting pieces of ivory, metal, or choice woods, or the like, into a groundwork of some other material, for ornamentation.

Insulated.—Detached from another building. A church is insulated, when not contiguous to any other edifice. A column is said to be insulated, when standing free from the wall; thus, the columns of peripteral temples were insulated.

Intaglio.—A sculpture or carving in which the figures are sunk below the general surface, such as a seal the impression of which in wax is in bas-relief; opposed to Cameo.

Intercolumniation.—The distance from column to column, the clear space between columns.

Interlaced Arches.—Arches where one passes over two openings, and they consequently cut or intersect each other.

Intrados.—Of an arch, the inner or concave curve of the arch stones.

Inverted Arches.—Those whose key-stone or brick is the lowest in the arch.

Ionic Order.—One of the orders of Classical architecture.

Iron Work.—In mediæval architecture, as an ornament, is chiefly confined to the hinges, etc., of doors and of church chests, etc. In some instances not only do the hinges become a mass of scroll work, but the surface of the doors is covered by similar ornaments. In almost all styles the smaller and less important doors had merely plain strap hinges, terminating in a few bent scrolls, and latterly in fleurs-de-lis. Escutcheon and ring handles, and the other furniture, partook more or less of the character of the time. On the Continent of Europe the knockers are very elaborate. At all periods doors have been ornamented with nails having projecting heads, sometimes square, sometimes polygonal, and sometimes ornamented with roses, etc. The iron work of windows is generally plain, and the ornament confined to simple fleur-de-lis heads to the stanchions. The iron work of screens enclosing tombs and chapels is noticed under *Grille, q. v.*

Jack.—An instrument for raising heavy loads, either by a crank, siren and pinion, or by hydraulic power, and in all cases worked by hand.

Jack Rafter.—A short rafter, used especially in hip-roofs.

Jamb.—The side-post or lining of a doorway or other aperture. The jambs of a window outside the frame are called Reveals.

Jamb-shafts.—Small shafts to doors and windows with caps and bases; when in the inside arris of the jamb of a window they are sometimes called Escon-sons.

Joggle.—A joint between two bodies so constructed by means of jogs or notches as to prevent their sliding past each other.

Joinery.—That branch in building confined to the nicer and more ornamental parts of carpentry.

Joist.—A small timber to which the boards of a floor or the laths of ceiling are nailed. It rests on the wall or on girders.

Keep.—The inmost and strongest part of a mediæval castle, answering to the citadel of modern times. The arrangement is said to have originated with Gundolf, the celebrated Bishop of Rochester. The Norman keep is generally a very massive square tower, the basement or stories partly below ground being used for stores and prisons. The main story is generally a great deal above ground level, with a projecting entrance, approached by a flight of steps and drawbridge. This floor is generally supposed to have been the guard-room or place for the soldiery; above this was the hall, which generally extended over the whole area of the building, and is sometimes separated by columns; above this are other apartments for the residents. There are winding staircases in the angles of the buildings, and passages and small chambers in the thickness of the walls. The keep was intended for the last refuge, in case the outworks were scaled and the other buildings stormed. There is generally a well in a mediæval keep, ingeniously concealed in the thickness of a wall, or in a pillar. The most celebrated of Norman times are the White Tower in London, the castles at Rochester, Arundel, and Newcastle, Castle Hedingham, etc. The keep was often circular.

Key-stone. The stone placed in the centre of the top of an arch. The character of the key-stone varies in different orders. In the Tuscan and Doric it is only a simple stone projecting beyond the rest; in the Ionic it is adorned with mouldings in the manner of a console; in the Corinthian and Composite it is a rich sculptured console.

King-post. The middle post of a trussed piece of framing for supporting the tie beam at the middle and the lower ends of the struts.

Knee. A piece of timber naturally or artificially bent to receive another to relieve a weight or strain.

Knob, Knot. The bunch of flowers carved on a corbel, or on a Boss.

Kremlin. The Russian name for the citadel of a town or city.

Label. Gothic: the drip or hood-moulding of an arch, when it is returned to the square.

Label Terminations. Carvings on which the labels terminate near the springing of the windows. In Norman times those were frequently grotesque heads of fish, birds, etc., and sometimes stiff foliage. In the Early English and Decorated periods they are often elegant knots of flowers, or heads of kings queens, bishops, and other persons supposed to be the founders of churches. In the Perpendicular period they are often finished with a short square, mitred return or knee, and the foliages are generally leaves of square or octagonal form.

Lacunar.—A panelled or coffered ceiling or soffit. The panels or caissons of a ceiling are by Vitruvius called lacunaria.

Lady-chapel.—A small chapel dedicated to the Virgin Mary, generally found in ancient cathedrals.

Lancet.—A high and narrow window pointed like a lancet, often called a lancet window

Landing.—A platform in a flight of stairs between two stories; the terminating of a stair.

Lantern.—A turret raised above a roof or tower and very much pierced, the better to transmit light. In modern practice this term is generally applied to any raised part in a roof or ceiling containing vertical windows, but covered in horizontally The name was also often applied to the louver or femerell on a roof to carry off the smoke, sometimes, too, to the open constructions at the top of towers, as at Ely Cathedral, probably because lights were placed in them at night to serve as beacons.

LACUNARS IN CEILING.

Lanterns of the Dead.—Curious small slender towers, found chiefly in the centre and west of France, having apertures at the top, where a light was exhibited at night to mark the place of a cemetery. Some have supposed that the round towers in Ireland may have served for this purpose

Lath.—A slip of wood used in slating, tiling, and plastering

Lattice.—Any work of wood or metal made by crossing laths, rods, or bars, and forming a net-work. 2. A reticulated window, made of laths or slips of iron, separated by glass windows, and only used where air rather than light is to be admitted, as in cellars and dairies.

Lavabo.—The lavatory for washing hands generally erected in cloisters of monasteries. A very curious one at Fontenay, surrounding a pillar, is given by Viollet le-Duc. In general, it is a sort of trough, and in some places has an almry for towels, etc.

Lavatory. A place for washing the person.

Lean-to. A small building whose rafters pitch or lean against another building or against a wall

Lectern.—The reading-desk in the choir of churches.

Ledge, or Ledgement. A projection from a plane, as slips on the side of window and door frames to keep them steady in their places

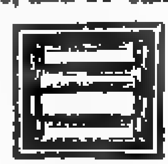
Ledgers. The horizontal pieces fastened to the standard poles or timbers of scaffolding raised around buildings during their erection Those which rest on the ledgers are called putlogs, and on these the boards are laid.

Lewis.—An iron clamp dovetailed into a large stone to lift it by

Lich-gate. A covered gate at the entrance of a cemetery, under the shelter of which the mourners rested with the corpse, while the procession of the clergy came to meet them There are several examples in England

Light.—A division or space in a sash for a single pane of glass; also a pane of glass

Linen Scroll. An ornament formerly used for filling panels, and so called from its resemblance to the convolutions of a folded napkin



LINEN SCROLL.

Lining.—Covering for the interior, as casing is covering the exterior surface of a building; also, such as linings of a door for windows, shutters, and similar work.

Lintel.—The horizontal piece which covers the opening of a door or window

Lip Mould. A moulding of the Perpendicular period like a hanging lip.

List, or Listel.—A little square moulding, to crown a larger; also termed a fillet.

Lithograph.—A print from a drawing on stone.

Lobby.—An open space surrounding a range of chambers, or seats in a theatre; a small hall or waiting room.

Lodge.—A small house in a park.

Loft. The highest room in a house, particularly if in the roof; also, a gallery raised up in a church to contain the rood, the organ, or singers.

Loggia.-An outside gallery or portico above the ground, and contained within the building.

Loop-hole. An opening in the wall of a building, very narrow on the outside, and splayed within, from which arrows or darts might be discharged on an enemy. They are often in the form of a cross, and generally have round holes at the ends.

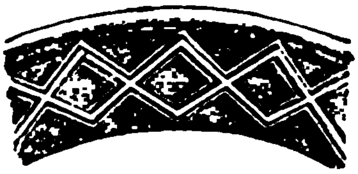
Lombard Architecture.—A name given to the round-arched architecture of Italy, introduced by the conquering Goths and Ostrogoths, and which superseded the Romanesque. It reigned between the eighth and twelfth centuries, during the time that the Saxon and Norman styles were in vogue in England, and corresponded with them in its development into the Continental Gothic.

Lotus.—A plant of great celebrity amongst the ancients, the leaves and blossoms of which generally form the capitals of Egyptian columns.

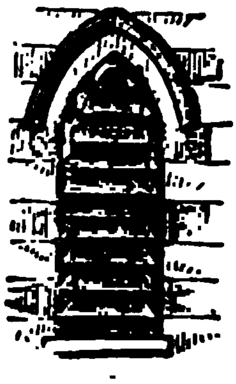
Louver. A kind of vertical window, frequently in the peaks of gables, and in the top of towers, and provided with horizontal slats which permit ventilation and exclude rain.

Lozenge Moulding. A kind of moulding used in Norman architecture, of many different forms, all of which are characterized by lozenge-shaped ornaments.

Lunette. The French term for the circular opening in the groining of the lower stories of towers, through which the bells are drawn up.



LOZENGE MOULDING.



LOUVER WINDOW.

Machicolation. A parapet or gallery projecting from the upper part of the wall of a house or fortification, supported by brackets or corbels, and perforated in the lower part so that the defenders of the building might throw down darts, stones, and sometimes hot sand, molten lead, etc., upon their assailants below.

Man-hole. A hole through which a man may creep into a drain, cesspool, steam-boiler, etc.

Manor-house. The residence of the suzerain or lord of the manor; in France the central tower or keep of a castle is often called the *manoir*.

Mansard Roof. Curb roof, invented by François Mansard, a distinguished French architect, who died in 1666.

Mansion. A residence of considerable size and pretension.

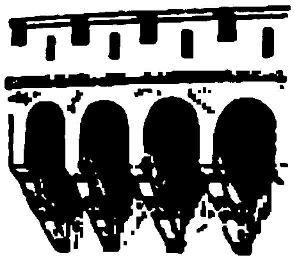
Mantel. The work over a fireplace in front of a chimney; especially, a shelf, usually ornamented, above the fireplace.

Marquetry. Inlaid work of fine hard pieces of wood of different colors, also of shells, ivory, and the like.

Mausoleum. A magnificent tomb or sumptuous sepulchral monument.

Medallion. Any circular tablet on which are embossed figures or busts.

Mediæval Architecture. The architecture of Eng-



MACHICOLATION.

land, France, Germany, etc., during the Middle Ages, including the *Norman* and Early Gothic styles. It comprises also the Romanesque, Byzantine and Saracenic, Lombard, and other styles.

Members.—The different parts of a building, the different parts of an entablature, the different mouldings of a cornice, etc.

Merlon.—That part of a parapet which lies between two embrasures.

Metope. The square recess between the triglyphs in a Doric frieze. It is sometimes occupied by sculptures.

Mezzanine.—A low story between two lofty ones. It is called by the French *entresol*, or inter-story.

Mezzo-rilievo.—Or mean relief, in comparison with alto-rilievo, or high relief.

Minaret. Turkish a circular turret rising by different stages or divisions, each of which has a balcony.

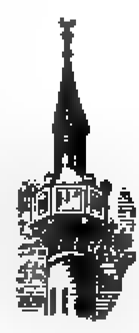
Minster.—Probably a corruption of monasterium—the large church attached to any ecclesiastical fraternity. If the latter be presided over by a bishop, it is generally called a Cathedral, if by an abbot, an Abbey, if by a prior, a Priory

METOPE.

Minute. The sixtieth part of the lower diameter of a column; it is the measure used by architects to determine the proportions of an order

Miserere. A seat in a stall of a large church made to turn up and afford support to a person in a position between sitting and standing. The under side is generally carved with some ornament, and very often with grotesque figures and caricatures of different persons.

Mitre.—A moulding returned upon itself at right angles is said to mitre. In joinery, the ends of any two pieces of wood of corresponding form, cut off at 45°, necessarily abut upon one another so as to form a right angle, and are said to mitre.



MINARET.

Modillion.—So called because of its arrangement in regulated distances; the enriched block or horizontal bracket generally found under the cornice of the Corinthian entablature. Less ornamented, it is sometimes used in the Ionic.

MODILLION.

Module.—This is a term which has been generally used by architects in determining the relative proportions of the various parts of a columnar ordinance. The semi-diameter of the column at its base is the module, which being divided into thirty parts called minutes, any part of the composition is said to be of so many modules and minutes, or minutes alone, in height, breadth, or projection The whole diameter is now generally preferred as a module, it being a better rule of proportion than its half.

Monastery. A set of buildings adapted for the reception of any of the various orders of monks, the different parts of which are described in the separate article, *Abbey*.

Monotriglyph.—The intercolumniations of the Doric order are determined by the number of triglyphs which intervene, instead of the number of diameters of the column, as in other cases, and this term designates the ordinary intercolumniation of one triglyph

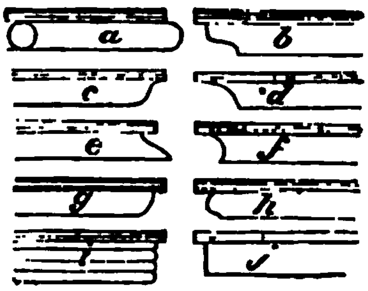
Monument.—A name given to a tomb, particularly to those fine structures recessed in the walls of mediæval churches.

Mosaic.—Pictorial representations, or ornaments, formed of small pieces of stone, marble, or enamel of various colors. In Roman houses the floors are often

entirely of mosaic, the pieces being cubical. The best examples of mosaic work are found in St. Mark's, at Venice.

Mosque.—A Mahometan temple, or place of worship.

Moulding. When any work is wrought into long regular channels or projections, forming curves or rounds, hollows, etc., it is said to be moulded, and each separate member is called a moulding. In mediæval architecture the principal mouldings are those of the arches, doors, windows, piers, etc. In the Early English style, the mouldings, for some time, formed groups set back in squares, and frequently very deeply undercut. The scroll moulding is also common. Small fillets now become very frequent in the keel moulding, from its resemblance in section to the bottom of a ship: sometimes, also, it has a peculiar hollow on each side, like two wings. Later in the Decorated style the mouldings are more varied in design, though hollows and rounds still prevail. The undercutting is not so deep, fillets abound, ogees are more frequent, and the wave mould, double ogee, or double ressaunt, is often seen. In many places the strings and labels are a round, the lower half of which is cut off by a plain chamfer. The mouldings in the later styles in some degree resemble those of the Decorated, flattened and extended; they run more into one another, having fewer fillets, and being, as it were, less grouped. One of the principal features of the change is the substitution of one, or perhaps two (seldom more), very large hollows in the set of mouldings. These hollows are neither circular nor elliptical, but obovate, like an egg cut across, so that one half is larger than the other. The brace mould also has a small bead, where the two ogees meet. Another sort of moulding, which has been called a lip mould, is common in parapets, bases, and weatherings.



MOULDINGS.

a, astragal; *b*, ogee; *c*, cymatium; *d*, cavetto; *e*, scotia, or casement; *f*, apophyges; *g*, ovolo, or quarter round; *h*, torus; *i*, reeding; *j*, band.

Mouldings, Ornamented. The Saxon and early Norman mouldings do not seem to have been much enriched, but the complete and later styles of Norman are remarkable for a profusion of ornamentation, the most usual of which is what is called the zigzag. This seems to be to Norman architecture what the meander or fret was to the Grecian; but it was probably derived from the Saxons, as it is very frequently found in their pottery. Bezants, quatrefoils, lozenges, crescents, billets, heads of nails, are very common ornaments. Besides these, battlements, cables; large ropes round which smaller ropes are turned, or, as our sailors say, "wormed"; scallops, pellets, chains, a sort of conical barrels, quaint stiff foliages, beaks of birds, heads of fishes, ornaments of almost every conceivable kind, are sculptured in Norman mouldings; and they are used in such profusion as has been attempted in no other style. The decorations on Early English mouldings are chiefly the dog tooth, which is one of the great characteristics of this style, though it is to be found in the Transition Norman. It is constantly placed in a deep hollow between two projecting mouldings, the dark shadow in the hollow contrasting in a very beautiful way with the light in these members. In this period and in the next the tympanum over doorways, particularly if they are double doors, is highly ornamented. Those of the Decorated period resemble the former, except that the foliage is more natural and the dog tooth gives way to the ball flower. Some of the hollows, also, are ornamented with rosettes at intervals, which are sometimes connected by a running tendril, as the ball flowers are frequently. Some very pleasing leaf like ornaments in the labels of windows are often found in Continental architecture. In the Perpen-

dicular period the mouldings are ornamented very frequently by square four-leaved flowers set at intervals, but the two characteristic ornaments of the time are running patterns of vine leaves, tendrils, and grapes in the hollows, which by old writers are called "vignettes in casements," and upright stiff leaves, generally called the Tudor leaf. On the Continent mouldings partook much of the same character.

Mullion, Munion.—The perpendicular pieces of stone, sometimes like columns, sometimes like slender piers, which divide the bays or lights of windows or screen-work from each other. In all styles, in less important work, the mullions are often simply plain chamfered, and more commonly have a very flat hollow on each side. In larger buildings there is often a bead or boutell on the edge, and often a single very small column with a capital. As tracery grew richer, the windows were divided by a larger order of mullion, between which came a lesser or subordinate set of mullions, which ran into each other. The term is also applied to a wood or iron division between two windows.

Multifoil.—A leaf ornament consisting of more than five divisions, applied to foils in windows.

Mutule.—The rectangular impending block under the corona of the Doric cornice, from which guttæ, or drops, depend. Mutule is equivalent to modillion, but the latter term is applied more particularly to enriched blocks or brackets, such as those of Ionic and Corinthian entablatures.

Narthex.—The long arcaded porch forming the entrance into the Christian basilica. Sometimes there was an inner narthex, or lobby, before entering the church. When this was the case, the former was called exo-narthex, and the latter eso-narthex. In the Byzantine churches this inner narthex forms part of the solid structure of the church, being marked off by a wall or row of columns, whereas in the Latin churches it was usually formed only by a wooden or other temporary screen.

Natural Beds.—In stratified rocks, is the surface of a stone as it lies in the quarry. If not laid in walls in their natural bed the laminæ separate.

Nave.—The central part between the arches of a church, which formerly was separated from a chancel or choir by a screen. It is so called from its fancied resemblance to a ship. In the nave were generally placed the pulpit and font. In continental Europe it often also contains a high altar, but this is of rare occurrence in England.

Necking.—The annulet or round, or series of horizontal mouldings, which separates the capital of a column from the plain part or shaft.

Newel.—In mediæval architecture, the circular ends of a winding staircase which stand over each other, and form a sort of cylindrical column.

Newel Post.—The post, plain or ornamented, placed at the first, or lowest step, to receive or start the hand-rail upon.

Niche.—A recess sunk in a wall, generally for the reception of a statue. Niches sometimes terminate by a simple label, but more commonly by a canopy, and with a bracket or corbel for the figure, in which case they are often called tabernacles.

Norman Style.—Was that species of Romanesque which was practised by the Normans, and which was introduced and fully developed in England after they had established themselves in it. The chief features of this style are plainness and massiveness. The arches, windows, and doorways were semicircular, the pillars were very massive, and often built up of small stones laid like brickwork.

Nosings.—The rounded and projecting edges of the treads of a stair, or the edge of a landing.

Obelisk.– Lofty pillars of stone, of a rectangular form, diminishing toward the top, and generally ornamented with inscriptions and hieroglyphics among the ancient Egyptians.

Observatory. A building erected on an elevated spot of ground for making astronomical observations.

Octostyle. A portico of eight columns in front.

Offsets.- When the face of a wall is not one continued surface, but sets in by horizontal jogs, as the wall grows higher and thinner, the jogs are called offsets.

Ogee.- -The name applied to a moulding, partly a hollow and partly a round, and derived no doubt from its resemblance to an O placed over a G. It is rarely found in Norman work, and is not very common in Early English. It is of frequent use in Decorated work, where it becomes sometimes double, and is called a wave moulding; and later still, two waves are connected with a small bead, which is then called a brace moulding. In ancient MSS. it is called a Ressaunt.

Orchestra. In ancient theatres, where the chorus used to dance; in modern theatres, where the musicians sit.

Order.- A column with its entablature and stylobate is so called. The term is the result of the dogmatic laws deduced from the writings of Vitruvius, and has been exclusively applied to those arrangements which they were thought to warrant.

Oriel Window. -Gothic: a projecting angular window, commonly of a trigonal or pentagonal form, and divided by mullions and transoms into different bays and compartments.

Orthography. A geometrical elevation of a building or other object in which it is represented as it actually exists or may exist, and not perspectively, or as it would appear.

Orthostyle. A columnar arrangement in which the columns are placed in a straight line.

Ovolo. Same as *Echinus*.

Pagoda. A name given to temples in India and China.

Palace. The dwelling of a king, prince, or bishop.

Pale. A fence picket, sharpened at the upper end.

Pane.- Probably a diminutive of panneau, a term applied to the different pieces of glass in a window; same as *Light*.

Panel. Properly a piece of wood framed within four other pieces of wood, as in the styles and rails of a door, filling up the aperture, but often applied both to the whole square frame and the sinking itself; also to the ranges of sunken compartments in wainscoting, cornices, corbel tables, groined vaults, ceilings, etc.

Pantograph. or **Pentagraph.** An instrument for copying on the same, or an enlarged or reduced scale.

Pantry. An apartment or closet in which bread and other provisions are kept

Papier-maché. A hard substance made of a pulp from rags or paper mixed with [illegible], and moulded into any desired shape. Much used for architectural ornaments.

Parapet. A dwarf wall along the edge of a roof, or round a terrace walk, etc., to prevent persons from falling over, and as a protection to the defenders in case of attack. Parapets are either plain, embattled, perforated, or panelled. The last two are found in all styles except the Norman. Plain parapets are simply portions of the wall generally overhanging a little, with coping at the top and corbel table below. Embattled parapets are sometimes panelled, but oftener

used for the discharge of arrows, etc. Perforated parapets are pierced in various vices as circles, trefoils, quatrefoils, and other designs—so that the light is in through. Panelled parapets are those ornamented by a series of panels, her oblong or square, and more or less enriched, but are not perforated. These common in the Decorated and Perpendicular periods

Pargeting.—A species of plastering decorated by impressing patterns on it when wet. These seem generally to have been made by sticking a number of pins in a board in certain lines or curves, and then pressing on the wet plaster in various directions, so as to form geometrical figures. Sometimes these devices are in relief, and in the time of Elizabeth represent figures, birds, foliages, etc. 2. Rough plastering, commonly adopted for the interior surface of chimneys.

Parlor.—A room in a house which the family usually occupy for society and conversation, and for receiving visitors. 2. The apartment in a monastery or nunnery where the inmates are permitted to meet and converse with each other, or with visitors and friends from without.

Parochial.—Belonging or relating to a parish.

Parquetry, or **Marquetry.**—A kind of inlaid floor composed of small pieces of wood either square or triangular, which are capable of forming, by their disposition, various combinations of figures; this description of joinery is very suitable for the floors of libraries, halls, and public apartments.

Party Walls.—Partitions of brick or stone between buildings on two adjoining properties.

Patera. A circular ornament resembling a dish, often worked in relief on friezes, etc.

Pavement.—Tessellated, a pavement of mosaic work, used by the ancients, made of square pieces of stone, etc., called Tessera.

Pavilion.—A turret or small insulated building, and comprised beneath a single roof; also, the projecting part in front of a building which marks the centre, and which sometimes flanks a corner, when it is termed an angular pavilion.

PATERA.

Pedestal.—The square support of a column, statue, etc.; and the base or lower part of an order of columns: it consists of a plinth for a base, the die, and a talon crowned for a cornice. When the height and width are equal, it is termed a square pedestal; one which supports two columns, a double pedestal; and if it supports a row of columns without any break, it is a continued pedestal

Pediment.—A low triangular crowning, ornamented, in front of a building, and over doors and windows. Pediments are sometimes made in the form of a segment; the space enclosed within the triangle is called the tympanum. Also, the gable ends of classic buildings, where the horizontal cornice is carried across the front, forming a triangle with the end of the roof.

Pendent.—A name given to an elongated boss, either moulded or foliated, such as hang down from the intersection of groins, especially in fan tracery, or at the end of hammer beams Sometimes long corbels, under the wall pieces, have been so called The name has also been given to the large masses depending from enriched ceilings, in the later works of the Pointed style.

Pendent Posts. A name given to those timbers which hang down the side of a wall from the plate in hammer beam trusses, and which receive the hammer braces.

Pendentive.—A name given to an arch which cuts off, as it were, the corners of a square building internally, so that the superstructure may become an octagon

or a dome. In mediæval architecture these arches, when under a spire in the interior of a tower, are called Squinches.

Pendentive Bracketing, or **Cove Bracketing.**—Springing from the rectangular walls of an apartment upward to the ceiling, and forming the horizontal part of the ceiling into a circle or ellipse.

Pentastyle.—Having five columns in front.

Pent-roof.—A roof with a slope on one side only.

Perch. A measure used in measuring stone work, being 24¾ cu. ft. and 16½ cu. ft., according to locality and custom.

Periptery. An edifice or temple surrounded by a peristyle.

Peristyle.—A range of columns encircling an edifice, such as that which surrounds the cylindrical drum under the cupola of St. Paul's. The columns of a Greek peripteral temple form a peristyle also, the former being a circular, and the latter a quadrilateral peristyle.

Perpendicular Style. The third and last of the Pointed or Gothic styles; also called the Florid style.

Perspective Drawing.—The art of making such a representation of an object upon a plane surface as shall present precisely the same appearance that the object itself would to the eye situated at a particular point.

Pews. A word of uncertain origin, signifying fixed seats in churches, composed of wood framing, mostly with ornamented ends. They seem to have come into general use early in the reign of Henry VI. and to have been rented and "well paid for" before the Reformation. Some bench ends are certainly of a decorated character, and some have been considered to be of the Early English period. They are sometimes of plain oak board, two and a half to three inches thick, chamfered, and with a necking and finial, generally called a poppy head; others are plainly panelled with bold cappings; in others the panels are ornamented with tracery or with the linen pattern, and sometimes with running foliages. The divisions are filled in with thin chamfered boarding, sometimes reaching to the floor, and sometimes only from the capping to the seat.

Picket.—A narrow board, often pointed, used in making fences; a pale or paling.

Pier-glass. A mirror hanging between windows.

Piers. The solid parts of a wall between windows, and between voids generally. The term is also applied to masses of brick-work or masonry which are insulated to form supports to gates or to carry arches, posts, girders, etc.

Pilasters. Are flat square columns, attached to a wall, behind a column, or along the side of a building, and projecting from the wall about a fourth or a sixth part of their breadth. The Greeks had a slightly different design for the capitals of pilasters, and made them the same width at top as at bottom, but the Romans gave them the same capitals as the columns, and made them of diminished width at the top, similar to the columns.

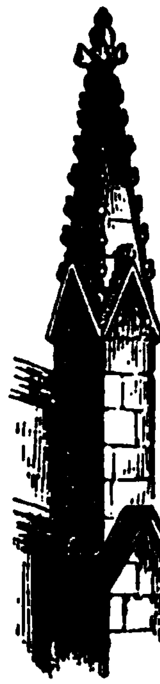
Pile. A large stake or trunk of a tree, driven into soft ground, as at the bottom of a river, or in made land, for the support of a building. (See p. 134.)

Pillar, or **Pyller.** A word generally used to express the round or polygonal piers, or those surrounded with clustered columns, which carry the main arches of a building. Saxon and Early Norman pillars are generally stout cylindrical shafts built up of small stones. Sometimes, however, they are quite square, sometimes with other squares breaking out of them (this is more common in French and German work), sometimes with angular shafts, and sometimes they are plain octagons. In Romanesque Norman work the pillar is sometimes square, with two or more semicircular or half columns attached. In the Early English period the pillars become loftier and lighter, and in most important buildings are a series

lustered columns, frequently of marble, placed side by side, sometimes set at rvals round a circular centre, and sometimes almost touching each other. se shafts are often wholly detached from the central pillar, though grouped nd it, in which case they are almost always of Purbeck or Bethersden marbles. Decorated work the shafts on plan are very often placed round a square set ;lewise, or a lozenge, the long way down the nave ; the centre or core itself is ten worked into hollows or other mouldings, to show betwen the shafts, and form part of the composition. In this and the latter part of the previous style ere is generally a fillet on the outer part of the shaft, forming what has been lled a keel moulding. They are also often, as it were, tied together by bands rmed of rings of stone and sometimes of metal. The small pillars at the jambs doors and windows, and in arcades, and also those slender columns attached pillars, or standing detached, are generally called shafts.

Pin.—A cylindrical piece of wood, iron, or steel, used to hold two or more pieces gether, by passing through a hole in each of them, as in a mortise and tenon int, or a pin joint of a truss.

Pinnacle.—An ornament originally forming the cap or crown of a buttress or all turret, but afterward used on parapets at the corners of wers and in many other situations. It was a weight to countert the thrust of the groining of roofs, particularly where there are flying buttresses ; it stopped the tendency to slip of the stone pings of the gables, and counterpoised the thrust of spires ; it rmed the piers to steady the elegant perforated parapets of ter periods ; and in France, especially, served to counterbalance e weight of overhanging corbel tables, huge gargoyles, etc. In e Early English period the smaller buttresses frequently finished ith gablets, and the more important with pinnacles supported ith clustered shafts. At this period the pinnacles were often pported on these shafts alone, and were open below ; and in rger work in this and the subsequent periods they frequently form ches and contain statues. In France, pinnacles, like spires, em to have been in use earlier than in England. There are small nnacles at the angles of the tower in the Abbey of Saintes. At oullet there are pinnacles in a similar position, each composed of ur small shafts, with caps and bases surmounted with small ramidal spires. In all these examples the towers have semicircular headed indows.



PINNACLE.

Pitch of a Roof.—The proportion obtained by dividing the span by the ight ; thus, we speak of its being one-half, one-third, one-fourth. When the ngth of the rafters is equal to the breadth of the building it is denominated othic.

Pitching-piece.—A horizontal timber, with one of its ends wedged into the ll at the top of a flight of stairs, to support the upper end of the rough strings.

Place.—An open piece of ground surrounded by buildings, generally decorated ith a statue, column, or other ornament.

Plan.—A horizontal geometrical section of the walls of a building ; or inditions, on a horizontal plane, of the relative positions of the walls and partitions, ith the various openings, such as windows and doors, recesses and projections, imneys and chimney-breasts, columns, pilasters, etc. This term is often inrrectly used in the sense of Design.

Planceer.—Is sometimes used in the same sense as soffit, but is more correctly plied to the soffit of the corona in a cornice.

Plastering.—A mixture of lime, hair, and sand, to cover lath-work between

timbers or rough walling, used from the earliest times, and very common in Roman work. In the Middle Ages, too, it was used not only in private, but in public constructions. On the inside face of old rubble walls it was not only used for purposes of cleanliness, rough work holding dirt and dust, but as a ground for distemper painting (tempera, or, as it is often improperly called, fresco), a species of ornament often used in the Middle Ages. At St. Albans Abbey, England, the Norman work is plastered, and covered with lines imitating the joints of stone. The same thing is found in English Perpendicular work. On the outside of rubble walls, and often of wood framing, it was used as roughcast; when ornamented in patterns outside, it is called pargeting.

Plate.—The piece of timber in a building which supports the end of the rafters.

Plinth.—The square block at the base of a column or pedestal. In a wall, the term plinth is applied to the projecting base or water table, generally at the level of the first floor.

Plumb.—Perpendicular; that is, standing according to a plumb line, as, the post of a house or wall is plumb.

Plumbing. The lead and iron pipes and other apparatus employed in conveying water, and for toilet purposes in a building; originally the art of casting and working in lead.

Ply. Used to denote the number of thicknesses of roofing paper, as three ply, four ply, etc.

Podium. A continued pedestal; a projection from a wall, forming a kind of gallery.

Polytriglyph.—An intercolumniation in the Doric order of more than two triglyphs.

Poppy Heads. Probably from the French *poupée*: the finials or other ornaments which terminate the tops of bench ends, either to pews or stalls. They are sometimes small human heads, sometimes richly carved images, knots of foliage, or finials, and sometimes fleurs-de-lis simply cut out of the thickness of the bench end and chamfered.

POPPY HEAD.

Porch. A covered erection forming a shelter to the entrance door of a large building. The earliest known are the long arcaded porches in front of the early Christian basilicas, called Narthex. In later times they assume two forms—one, the projecting erection covering the entrance at the west front of cathedrals, and divided into three or more doorways, etc.; and the other, a kind of covered chambers open at the ends, and having small windows at the sides as a protection from rain.

Portal. A name given to the deeply recessed and richly decorated entrance doors to the cathedrals in Continental Europe.

Portcullis. A strong framed grating of oak, the lower points shod with iron, and sometimes entirely made of metal, hung so as to slide up and down in grooves with counterbalances, and intended to protect the gateways of castles, etc.

Portico. An open space before the door or other entrance to any building, fronted with columns. A portico is distinguished as prostyle or *in antis* according as it projects from or recedes within the building, and is further designated by the number of columns its front may consist of.

Post. Square timbers set on end. The term is especially applied to those [illegible] the corners of a building, and are framed into bressummers or cross beams under the walls.

Posticum. A portico behind a temple.

Presbytery. A word applied to various parts of large churches in a very an

iguous way. Some consider it to be the choir itself; others, what is now named he sacrarium. Traditionally, however, it seems to be applied to the vacant pace between the back of the high altar and the entrance to the lady-chapel, as t Lincoln and Chichester; in other words, the back- or retro-choir.

Priming.—The laying on of the first shade of color, in oil paint, and generally onsisting mostly of oil, to protect and fill the wood.

Priory.—A monastic establishment, generally in connection with an abbey, nd presided over by a prior, who was a subordinate to the abbot, and held much he same relation to that dignitary as a dean does to a bishop.

Profile.—The outline; the contour of a part, or the parts composing an order, s of a base, cornice, etc.; also, the perpendicular section. It is in the just proortion of their profiles that the chief beauties of the different orders of archiecture depend. The ancients were most careful of the profiles of their mouldngs.

Proscenium.—The front part of the stage of ancient theatres, on which the ctors performed.

Prostyle.—A portico in which the columns project from the building to which t is attached.

Protractor.—A mathematical instrument for laying down and measuring angles on paper, used in drawing or plotting.

Pseudo-dipteral.—False double-winged. When the inner row of columns of a dipteral arrangement is omitted and the space from the wall of the building o the columns is preserved, it is pseudo-dipteral.

Puddle.—To settle loose dirt by turning on water, so as to render it firm and solid.

Pugging.—A coarse kind of mortar laid on the boarding, between floor joists, o prevent the passage of sound; also called deafening.

Pulpit.—A raised platform with enclosed front, whence sermons, homilies, etc., were delivered. Pulpits were probably derived in their modern form from the ambones in the early Christian church. There are many old pulpits of stone, though the majority are of wood. Those in the churches are generally hexagonal or octagonal; and some stand on stone bases, and others on slender wooden stems, like columns. The designs vary according to the periods in which they were erected, having panelling, tracery, cuspings, crockets, and other ornaments then in use. Some are extremely rich, and ornamented with color and gilding. A few also have fine canopies or sounding boards. Their usual place is in the nave, mostly on the north side, against the second pier from the chancel arch. Pulpits for addressing the people in the open air were common in the Mediæval period, and stood near a road or cross. Thus, there was one at Spitalfields, and one at St. Paul's, London. External pulpits still remain at Magdalen College, Oxford, and at Shrewsbury, England.

Purlins.—Those pieces of timbers which support the rafters to prevent them from sinking.

Putlog.—Horizontal pieces for supporting the floor of a scaffold, one end being inserted into putlog holes, left for that purpose in the masonry.

Putty in Plastering.—Lump lime slacked with water to the consistency of cream, and then left to harden by evaporation till it becomes like soft putty. It is then mixed with plaster of Paris, or sand, for the finishing coat.

Puzzolana.—A grayish earth used for building under water.

Pyramid.—A solid, having one of its sides, called a base, a plane figure, and the other sides triangles, these points joining in one point at the top, called the vertex. Pyramids are called triangular, square, etc., according to the form of their bases.

Pyx.—In Roman Catholic churches, the box in which the host, or consecrated wafer, is kept.

Quadrangle.—A square or quadrangular court surrounded by buildings, as was often done formerly in monasteries, colleges, etc.

Quarry. A pane of glass cut in a diamond or lozenge form.

Quarry-face.—Ashlar as it comes from the quarry, squared off for the joints only, with split face. In distinction from Rock-face, in that the latter may be weather-worn, while Quarry-face should be fresh split. The terms are often used indiscriminately.

Quatrefoil. Any small panel or perforation in the form of a four-leaved flower. Sometimes used alone, sometimes in circles and over the aisle windows, but more frequently in square panels. They are generally cusped, and the cusps are often feathered.

Queen Truss. A truss framed with two vertical tie-posts, in distinction from the king-post, which has but one. The upright ties are called Queen-posts.

Quirk Mouldings.—The convex part of Grecian mouldings when they recede at the top, forming a reëntrant angle, with the surface which covers the mouldings.

Quoins. Large squared stones at the angles of buildings, buttresses, etc., generally used to stop the rubble or rough stone work, and that the angles may be true and stronger. Saxon quoin stones are said to have been composed of one long and one short stone alternately. Early quoins are generally roughly axed; in later times they had a draught tooled by the chisel round the outside edges, and later still were worked fine from the saw.

Rafters. The joist to which the roof boarding is nailed. *Principal rafters* are the upper timbers in a truss, having the same inclination as the common rafters.

Rail. A piece of timber or metal extending from one post to another, as in fences, balustrades, staircases, etc. In framing and panelling, the horizontal pieces are called rails, and the perpendicular, *stiles*.

Raking. Mouldings whose arrises are inclined to the horizon.

Ramp. A concavity on the upper side of hand railings formed over risers, made by a sudden rise of the steps above. Any concave bend or slope in the cap or upper member of any piece of ascending or descending workmanship.

Rampant. A term applied to an arch whose abutments spring from an inclined plane.

Random Work. A term used by stone-masons for stones fitted together at random without any attempt at laying them in courses. *Random Coursed Work* is a like term applied to work coursed in horizontal beds, but the stones are of any height, and fitted to one another.

Range Work. Ashlar laid in horizontal courses: same as coursed ashlar.

Rebate. A groove on the edges of a board.

Recess. A depth of some inches in the thickness of a wall, as a niche, etc.

Refectory. The hall of a monastery, convent, etc., where the religious took their chief meals together. It much resembled the great halls of mansions, castles, etc., except that there frequently was a sort of ambo, approached by steps, from which to read the Legenda Sanctorum, etc., during meals.

Reglet. A flat, narrow moulding, used to separate from each other the parts or members of compartments and panels, to form frets, knots, etc.

Renaissance a new birth. A name given to the revival of Roman architecture which sprang into existence in Italy as early as the beginning of the fifteenth

ury, and reached its zenith in that country at the close of the century. There everal divisions of this style as developed in different localities; viz.,

ie *Florentine Renaissance*, of which the Pitti Palace, by Brunelleschi, is one ie best examples.

ie *Venetian Renaissance*, characterized by its elegance and richness.

ie *Roman Renaissance*, which originated in Rome, under the architects vn as Bronte, Vignola, and Michael Angelo. Of this style the Farnese Palace, 'eter's, and the modern Capitol at Rome are the best examples.

ie *French Renaissance*, introduced into France in the latter part of the fifth century, by Italian architects, where it flourished until the middle of the nteenth century. The Renaissance style was introduced into Germany about middle of the sixteenth century, and into England about the same time by ı of Padua, architect to Henry VIII. This style in England is generally vn under the name of Elizabethan.

endering.—In drawing, finishing a perspective drawing in ink or color, to g out the spirit and effect of the design. 2. The first coat of plaster on brick :one work.

eredos, Dorsal, or **Dossel.**—The screen or other ornamental work at the back n altar. In some large English cathedrals, as Winchester, Durham, St. Albans, , this is a mass of splendid tabernacle work, reaching nearly to the groining. maller churches there are sometimes ranges of arcades or panellings behind ıltars; but, in general, the walls at the back and sides of them were of plain onry, and adorned with hangings or paraments. In the large churches of tinental Europe the high altar usually stands under a sort of canopy or ciboı, and the sacrarium is hung round at the back and sides with curtains on 'able rods.

eticulated Work.—That in which the courses are arranged in a form like meshes of a net. The stones or bricks are square and placed lozenge-.

eturn.—The continuation of a moulding, projection, etc., in an opposite ction.

eturn Head.—One that appears both on the face and edge of a work.

eveal.—The two vertical sides of an aperture, between the front of a wall the window or door frame.

ib.—A moulding or projecting piece upon the interior of a vault, or used to ı tracery and the like. The earliest groining had no ribs. In early Norman s plain flat arches crossed each other, forming ogive ribs. These by degrees ıme narrower, had greater projection, and were chamfered. In later Norı work the ribs were often formed of a large roll placed upon the flat band, then of two rolls side by side with a smaller roll or a fillet between them, h like the lower member. Sometimes they are enriched with zigzags and r Norman decorations, and about this time bosses became of very general As styles progressed, the mouldings were more undercut, richer, and more orate, and had the dog-tooth or ball-flower or other characteristic ornament ıe hollows. In all instances the mouldings are of similar contours to those rches, etc., of the respective periods. Later, wooden roofs are often formed cants or polygonal barrel vaults, and in these the ribs are generally a cluster ounds, and form square or stellar panels, with carved bosses or shields at the rsections.

idge.—The top of a roof which rises to an acute angle.

idge-pole.—The highest horizontal timber in a roof, extending from top to of the several pairs of rafters of the trusses, for supporting the heads of the : rafters.

Rilievo, or Relief.—The projection of an architectural ornament.

Rise.—The distance through which anything rises, as the rise of a stair, or inclined plane.

Riser.—The vertical board under the tread in stairs.

Rococo Style.—A name given to that variety of the Renaissance which was in vogue during the seventeenth and the latter part of the sixteenth century.

Romanesque Style.—The term Romanesque embraces all those styles of architecture which prevailed between the destruction of the Roman Empire and the beginning of Gothic architecture. In it are included the Early Roman Christian architecture, Byzantine, Mahometan, and the later Romanesque architecture proper, which was developed in Italy, France, England, and Germany. This later Romanesque, which was quite different from the preceding, came into vogue during the tenth century, and reached its height during the twelfth century, and in the thirteenth century gave way to the Pointed or Gothic style. In England, Romanesque architecture is known under the name of the Saxon, Norman, and Lombard styles, according to the different political periods.

Rood.—A name applied to a crucifix, particularly to those which were placed in the rood-loft or chancel screens. These generally had not only the image of the crucified Saviour, but also those of St. John and the Virgin Mary standing one on each side. Sometimes other saints and angels are by them, and the top of the screen is set with candlesticks or other decoration.

Rood-loft, Rood-screen, Rood-beam, Jube Gallery, etc.—The arrangement to carry the crucifix or rood, and to screen off the chancel from the rest of the church during the breviary services, and as a place whence to read certain parts of those services. Sometimes the crucifix is carried simply on a strong transverse beam, with or without a low screen, with folding-doors below but forming no part of such support. In European churches the general construction of wooden screens is close panelling beneath, about 3 feet to 3 feet 6 inches high, on which stands screen work composed of slender turned balusters or regular wooden mullions, supporting tracery more or less rich, with cornices, cresting, etc., and often painted in brilliant colors and gilded. These not only enclose the chancels, but also chapels, chantries, and sometimes even tombs. In English mansions, and some private houses, the great halls were screened off by a low passage at the end opposite to the dais, over which was a gallery for the use of minstrels or spectators. These screens were sometimes close and sometimes glazed.

Rood-tower.—A name given by some writers to the central tower, or that over the intersection of the nave and chancel with the transepts.

Roof.—The covering or upper part of any building.

Roofing.—The material put on a roof to make it water tight.

Rose Window.—A name given to a circular window with radiating tracery; called also wheel window.

Rostrum.—An elevated platform from which a speaker addresses an audience.

Rotunda.—A building which is round both within and without. 2. A circular room under a dome in large buildings is also called the rotunda.

Roughcast.—A sort of external plastering in which small sharp stones are mixed, and which, when wet, is forcibly thrown or cast from a trowel against the wall, to which it forms a coating of pleasing appearance. Roughcast work has been used in Europe for several centuries, where it was much used in timber houses, and when well executed the work is sound and durable. The mortar for roughcast work should always have cement mixed with it.

Rubble Work.—Masonry of rough, undressed stones. When only the roughest irregularities are knocked off, it is called scabbled rubble, and when the

stones in each course are rudely dressed to nearly a uniform height, ranged rubble.

Rudenture.—The figure of a rope or staff, which is frequently used to fill up the flutings of columns, the convexity of which contrasts with the concavity of the flutings, and serves to strengthen the edges. Sometimes, instead of a convex shape, the flutings are filled with a flat surface; sometimes they are ornamentally carved, and sometimes on pilasters, etc. Rudentures are used in relief without flutings, as their use is to give greater solidity to the lower part of the shaft, and secure the edges. They are generally only used in columns which rise from the ground and are not to reach above one-third of the height of the shaft.

Rustic or **Rock Work.**—A mode of building in imitation of nature. This term is applied to those courses of stone work the face of which is jagged or picked so as to present a rough surface. That work is also called rustic in which the horizontal and vertical channels are cut in the joinings of stones, so that when placed together an angular channel is formed at each joint. *Frosted rustic work* has the margins of the stones reduced to a plane parallel to the plane of the wall, the intermediate parts having an irregular surface. *Vermiculated rustic work* has these intermediate parts so worked as to have the appearance of having been eaten by worms. *Rustic chamfered work*, in which the face of the stones is smooth, and parallel to the face of the wall, and the angles bevelled to an angle of one hundred and thirty-five degrees with the face so that two stones coming together on the wall, the bevelling will form an internal right angle.

Sacristy.—A small chamber attached to churches, where the chalices, vestments, books, etc., were kept by the officer called the sacristan. In the early Christian basilicas there were two semicircular recesses or apsides, one on each side of the altar. One of these served as a sacristy, and the other as the bibliotheca or library. Some have supposed the sacristy to have been the place where the vestments were kept, and the vestry that where the priests put them on; but we find from Durandus that the sacrarium was used for both these purposes. Sometimes the place where the altar stands enclosed by the rails has been called sacrarium.

Saddle Bars.—Narrow horizontal iron bars passing from mullion to mullion, and often through the whole window, from side to side, to steady the stone work, and to form stays, to which the lead work is secured. When the bays of the windows are wide, the lead lights are further strengthened by upright bars passing through eyes forged on the saddle bars, and called stanchions. When saddle bars pass right through the mullions in one piece, and are secured to the jambs, they have sometimes been called stay bars.

Sagging.—The bending of a body in the middle by its own weight, or the load upon it.

Salient.—A projection.

Salon.—A spacious and elegant apartment for the reception of company, or for state purposes, or for the reception of paintings, and usually extending through two stories of the house. It may be square, oblong, polygonal, or circular.

Sanctuary.—That part of a church where the altar is placed; also, the most sacred or retired part of a temple. 2. A place for divine worship; a church.

Sanctus Bell-cot, or **Turret.**—A turret or enclosure to hold the small bell sounded at various parts of the service, particularly where the words "Sanctus," etc., are read. This differs but little from the common bell cot, except that it is generally on the top of the arch dividing the nave from the chancel. Sometimes,

however, the bell seems to have been placed in a cot outside the wall. In England sanctus bells have also been placed over the gables of porches. In Continental Europe they run up into a sort of small slender spire, called *flèche* in France, and *guglio* in Italy.

Saracenic Architecture.—That Eastern style employed by the Saracens, and which distributed itself over the world with the religion of Mahomet. It is a modification and combination of the various styles of the countries which they conquered.

Sarcophagus. A tomb or coffin made of stone, and intended to contain the body.

Sash.—The framework which holds the glass in a window.

Scagliola. An imitation of colored marbles in plaster work, made by a combination of gypsum, glue, isinglass, and coloring matter, and finished with a high polish, invented between 1600 and 1649.

Scabble.—To dress off the rougher projections of stones for rubble masonry with a stone axe or scabbling hammer.

Scantling. The dimensions of a piece of timber in breadth and thickness; also, studding for a partition, when under five inches square.

Scarfing. The joining and bolting of two pieces of timber together transversely, so that the two appear as one.

Sconce.- A fixed hanging or projecting candlestick.

Scotia.—A concave moulding, most commonly used in bases, which projects a deep shadow on itself, and is thereby a most effective moulding under the eye, as in a base. It is like a reversed ovolo, or, rather, what the mould of an ovolo would present.

Scratch Coat.—The first coat of plaster, which is scratched to afford a bond for the second coat.

Screeds. Long narrow strips of plaster put on horizontally along a wall, and carefully faced out of wind, to serve as guides for plastering the wide intervals between them.

Screen. Any construction subdividing one part of a building from another, as a choir, chantry, chapel, etc. The earliest screens are the low marble podia shutting off the chorus cantantium in the Roman basilicas, and the perforated cancelli enclosing the bema, altar, and seats of the bishops and presbyters. The chief screens in a church are those which enclose the choir or the place where the breviary services are recited. In Continental Europe this is done not only by doors and screen work, but also, when these are of open work, by curtains, the laity having no part in these services. In England screens were of two kinds: one, of open wood-work, generally called rood-screens or jubes, and which the French call *grilles, clôtures du chœur*; the other, massive enclosures of stone work enriched with niches, tabernacles, canopies, pinnacles, statues, crestings, etc., as at Canterbury, York, Gloucester, and many other places.

Scribing. Fitting wood-work to an irregular surface.

Section. A drawing showing the internal heights of the various parts of a building. It supposes the building to be cut through entirely, so as to exhibit the walls, the heights of the internal doors and other apertures, the heights of the stories, thicknesses of the floors, etc. It is one of the species of drawings necessary to the exhibition of a Design.

Sedilia. Seats used by the celebrants during the pauses in the mass. They are generally three in number for the priest, deacon, and sub-deacon—and are in England almost always a species of niches cut into the south walls of churches, separated by shafts or by a species of mullions, and crowned with canopies, pinnacles, and other enrichments more or less elaborate. The piscina and

ambry sometimes are attached to them. In Continental Europe the sedilia are often movable seats; a single stone seat has rarely been found.

Set-off.—The horizontal line shown where a wall is reduced in thickness, and, consequently, the part of the thicker portion appears projecting before the thinner. In plinths this is generally simply chamfered. In other parts of work the set-off is generally concealed by a projecting string. Where, as in parapets, the upper part projects before the lower, the break is generally hid by a corbel table. The portions of buttress caps which recede one behind another are also called set-offs.

Shaft.—In Classical architecture that part of a column between the necking and the apophyge at the top of the base. In later times the term is applied to slender columns either standing alone or in connection with pillars, buttresses, jambs, vaulting, etc.

Shed Roof, or **Lean-to.**—A roof with only one set of rafters, falling from a higher to a lower wall, like an aisle roof.

Shore.—A piece of timber placed in an oblique direction to support a building or wall temporarily while it is being repaired or altered.

Shrine.—A sort of ark or chest to hold relics. It is sometimes merely a small box, generally with a raised top like a roof; sometimes an actual model of churches; sometimes a large construction, like that of Edward the Confessor at Westminster, of St. Genevieve at Paris, etc. Many are covered with jewels in the richest way; that of San Carlo Borromeo, at Milan, is of beaten silver.

Sills.—Are the timbers on the ground which support the posts and superstructure of a timber building. The term is most frequently applied to those pieces of timber or stone at the bottom of doors or windows.

Skewback.—The inclined stone from which an arch springs.

Skirtings.—The narrow boards which form a plinth around the margin of a floor, now generally called the base.

Sleeper.—A piece of timber laid on the ground to receive floor joists.

Soffit.—The lower horizontal face of anything, as, for example, of an entablature resting on and lying open between the columns or the under face of an arch where its thickness is seen.

Sound Board.—The covering of a pulpit to deflect the sound into a church.

Spall.—Bad or broken brick: stone chips.

Span.—The distance between the supports of a beam, girder, arch, truss, etc.

Spandrel, or **Spandril.**—The space between any arch or curved brace and the level label, beams, etc., over the same. The spandrels over doorways in Perpendicular works are generally richly decorated.

Specification.—Architect's. The designation of the kind, quality, and quantity of work and material to go in a building, in conjunction with the working drawings.

Spire.—A sharply pointed pyramid or large pinnacle, generally octagonal in England, and forming a finish to the tops of towers. Timber spires are very common in England. Some are covered with lead in flat sheets, others with the same metal in narrow strips laid diagonally. Very many are covered with shingles. In Continental Europe there are some elegant examples of spires of open timber work covered with lead.

Splayed.—The jamb of a door, or anything else of which one side makes an oblique angle with the other.

Springer.—The stone from which an arch springs: in some cases this is a capital, or impost; in other cases the mouldings continue down the pier. The lowest stone of the gable is sometimes called a springer.

Squinches.—Small arches or corbelled set-offs running diagonally and, as it

were, cutting off the corners of the interior of towers, to bring them from the square to the octagon, etc., to carry the spire.

Squint.—An oblique opening in the wall of a church: especially, in mediæval architecture, an opening so placed as to afford a view of the high altar from the transept or aisles.

Staging. A structure of posts and boards for supporting workmen and material in building.

Stall.—A fixed seat in the choir for the use of the clergy. In early Christian times the thronus cathedra, or seat of the bishop, was in the centre of the apsis or bema behind the altar, and against the wall; those of the presbyters also were against the wall, branching off from side to side around the semicircle. In later times the stalls occupied both sides of the choir, return seats being placed at the ends for the prior, dean, precentor, chancellor, or other officers. In general, in cathedrals, each stall is surmounted by tabernacle work, and rich canopies, generally of oak.

Stanchion.—A word derived from the French *étançon*, a wooden post, applied to the upright iron bars which pass through the eyes of the saddle bars or horizontal irons to steady the lead lights. The French call the latter *traverses*, the stanchions *montants*, and the whole arrangement *armature*. Stanchions frequently finish with ornamental heads forged out of the iron.

Steeple.—A general name for the whole arrangement of tower, belfry, spire, etc.

Stereobate.—A basement, distinguished from the nearly equivalent term stylobate by the absence of columns.

Stile.—The upright piece in framing or panelling.

Stilted.—Anything raised above its usual level. An arch is stilted when its centre is raised above the line from which the arch appears to spring.

Stoop. A seat before the door; often a porch with a balustrade and seats on the sides.

Stoup. A basin for holy water at the entrance of Roman Catholic churches, into which all who enter dip their fingers and cross themselves.

Straight Arch.—A form of arch in which the intrados is straight, but with its joints radiating as in a common arch.

Strap. An iron plate for connecting two or more timbers, to which it is screwed by bolts. It generally passes around one of the timbers.

Stretcher. A brick or block of masonry laid lengthwise of a wall.

String Board. A board placed next to the well-hole in wooden stairs, terminating the ends of the steps. The string piece is the piece of board put under the treads and risers for a support, and forming the support of the stair.

String-course.—A narrow, vertically faced and slightly projecting course in an elevation. If window-sills are made continuous, they form a string-course; but if this course is made thicker or deeper than ordinary window-sills, or covers a set-off in the wall, it becomes a blocking-course. Also, horizontal mouldings running under windows, separating the walls from the plain part of the parapets, dividing towers into stories or stages, etc. Their section is much the same as the labels of the respective periods; in fact, these last, after passing round the windows, frequently run on horizontally and form strings. Like labels, they are often decorated with foliages, ball flowers, etc.

Studs, or **Studding.** The small timbers used in partitions and outside wooden walls, to which the laths and boards are nailed.

Style. The term style in architecture has obtained a conventional meaning beyond its simpler one, which applies only to columns and columnar arrangements. It is now used to signify the differences in the mouldings, general out-

naments, and other details which exist between the works of various and also those differences which are found to exist between the works ition at different times.

ate.—A basement to columns. Stylobate is synonymous with pedestal, plied to a continued and unbroken substructure or basement to columns, latter term is confined to insulated supports. The Greek temples gen- d three or more steps all around the temple, the base of the column n the top step ; this was the stylobate.

lium.—A name sometimes given to the seat in the stalls of churches ; miserere.

er.—A girder or main-beam of a floor ; if supported on two-story posts below, it is called a Brace-summer.

se.—A cornice or series of mouldings on the top of the base of a pedes um, etc.; a moulding above the base.

e.—To make plane and smooth.

e.—An intercolumniation to which two diameters are assigned.

nacle.—A species of niche or recess in which an image may be placed. generally highly ornamented and often surmounted with crocketed The word tabernacle is also often used to denote the receptacle for relics, as often made in the form of a small house or church.

nacle Work.—The rich ornamental tracery forming the canopy, etc., rnacle, is called tabernacle work ; it is common in the stalls and screens lrals, and in them is generally open or pierced through.

Trimmer.—A trimmer next to the wall, into which the ends of joists are to avoid flues.

—To pound the earth down around a wall after it has been thrown in.

try.—A kind of woven hangings of wool or silk, ornamented with figures, formerly to cover and adorn the walls of rooms. They were often of costly materials and beautifully embroidered.

e.—An edifice destined, in the earliest times, for the public exercise of worship.

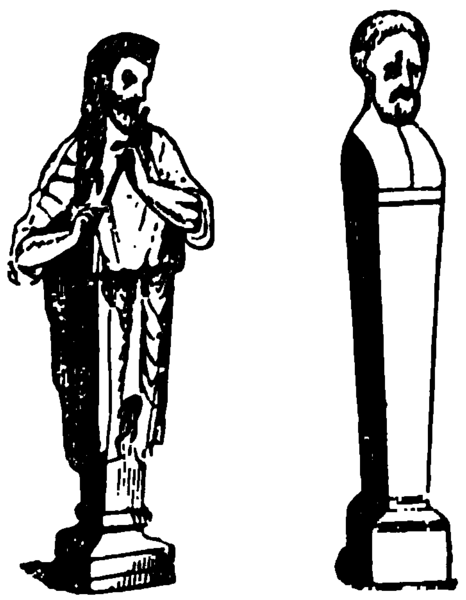
et, or Template.—A mould used by masons for cutting or setting . A short piece of timber sometimes laid under a girder.

nal.—Figures of which the upper parts only, or perhaps the head and alone, are carved, the rest running into opiped, and sometimes into a diminishing with feet indicated below, or even with- , are called terminal figures.

-cotta.—Baked clay of a fine quality. ed for bas-reliefs for adorning the friezes es. In modern times employed for archi- ornaments, statues, vases, etc.

lated Pavements.—Those formed of or, as some write it, tesselIæ, or small om half an inch to an inch square, like pottery, stone, marble, enamel, etc.

style.—A portico of four columns in



ANCIENT TERMINI.

bate.—That on which a dome or cupola This is a term not in general use, but it is ess of useful application. What is generally termed the attic above the and under the cupola of St. Paul's, London, would be correctly desig-

nated the tholobate. A tholobate of a different description, and one to which no other name can well be applied, is the circular substructure to the cupola of the University College, London.

Throat. A channel or groove made on the under-side of a string-course, coping, etc., to prevent water from running inward toward the walls.

Tie. A timber, rod, chain, etc., binding two bodies together, which have a tendency to separate or diverge from each other. The *tie-beam* connects the bottom of a pair of principal rafters, and prevents them from bursting out the wall.

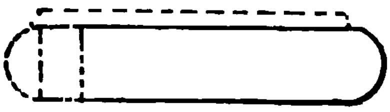
Tiles. Flat pieces of clay burned in kilns, to cover roofs in place of slates or lead. 2. Also, flat pieces of burned clay, either plain or ornamented, glazed or unglazed, used for floors, wainscoting, and about fireplaces, etc. 3. Small square pieces of marble are also called tile.

Tongue. The part of a board left projecting, to be inserted into a groove.

Tooth Ornament. One of the peculiar marks of the Early English period of Gothic architecture, generally inserted in the hollow mouldings of doorways, windows, etc.

Torso. A mutilated statue of which nothing remains but the trunk. Columns with twisted shafts have also this term. Of this kind there are several varieties.

Torus. A protuberance or swelling, a moulding whose form is convex, and generally nearly approaches a semicircle. It is most frequently used in bases, and is generally the lowest moulding in a base.



TORUS.

Tower. An elevated building originally designed for purposes of defence. Those buildings are of the remotest antiquity, and are, indeed, mentioned in the earliest Scriptures. In mediaeval times they were generally attached to churches, to cemeteries, to castles, or used as bell-towers in public places of large cities. In churches, the towers of the Saxon period were generally square. Norman towers were also generally square. Many were entirely without buttresses; others had broad, flat, shallow projections which served for this purpose. The lower windows were very narrow, with extremely wide splays inside, probably intended to be defended by archers. The upper windows, like those of the preceding style, were generally separated into two lights, but by a shaft or short column, and not by a baluster. Early English towers were generally taller, and of more elegant proportions. They almost always had large projecting buttresses, and frequently stone staircases. The lower windows, as in the former style, were frequently mere arrow slits; the upper were in couplets or triplets, and sometimes the tower top had an arcade all round. The spires were generally broach spires; but sometimes the tower tops finished with corbel courses and plain parapets, and (rarely) with pinnacles. There are a few Early English towers which break into the octagon from the square toward the top, and still fewer which finish with two gables. Both these methods of termination, however, are common in Continental Europe. At Vendôme, Chartres, and Senlis the towers have octagonal upper stages surrounded with pinnacles, from which elegant spires arise. In the North of Italy, and in Rome, they are generally tall square shafts in four to six stages, without buttresses, with couplets or triplets of semicircular windows in each stage, generally crenellated at top, and covered with a low pyramidal roof. The well known leaning tower at Pisa is cylindrical, in five stories of arcaded colonnades. In Ireland there are in some of the churchyards very curious round towers.

Tracery. The ornamental filling in of the heads of windows, panels, circular windows, etc., which has given such characteristic beauty to the architecture of

enth century. Like almost everything connected with mediæval archi- his elegant and sometimes fairy-like decoration seems to have sprung smallest beginnings. The circular-headed window of the Normans gave way to the narrow-pointed lancets of the Early English period, ss light was afforded by the latter system than by the former, it was to have a greater number of windows; and it was found convenient to em together in couplets, triplets, etc. When these couplets were l under one label, a sort of vacant space or spandrel was formed over s and under the label. To relieve this, the first attempts were simply to this flat spandrel, first by a simple lozenge-shaped or circular opening, ward by a quatrefoil. By piercing the whole of the vacant spaces in w head, carrying mouldings around the tracery, and adding cusps to it, tion of tracery was complete, and its earliest result was the beautiful al work such as is found at Westminster Abbey.

pt.—That portion of a church which passes transversely between the choir at right angles, and so forms a cross on the plan.

m.—The horizontal construction which divides a window into heights . Transoms are sometimes simple pieces of mullions placed trans- cross-bars, and in later times are richly decorated with cuspings,

se.—To plane in a direction across the grain of the wood, as to traverse planing across the boards.

—The horizontal part of a step of a stair.

.—A cusping the outline of which is derived from a three-leaved flower the quatrefoil and cinque-foil are from those with four and five.

.—Lattice-work of metal or wood for vines to run on.

.—A movable frame or support for anything; when made of a cross four legs it is called by carpenters a horse.

um.—The arcaded story between the lower range of piers and arches lere-story. The name has been supposed to be derived from *tres* and ree doors, or openings—that being a frequent number of arches in each

ph.—The vertically channelled tablets of the Doric frieze are called because of the three angular channels in them—two perfect and one the two chamfered angles or hemiglyphs being reckoned as one. The nk spaces between the triglyphs on a frieze are called metopes.

-Of a door, sometimes used to denote the locks, knobs, and hinges.

er.—The beam or floor joist into which a header is framed.

er Arch.—An arch built in front of a fireplace, in the thickness of the ween two trimmers. The bottom of the arch starting from the chimney p pressing against the header.

ointing.—Marking the joints of brickwork with a narrow paralle ine putty.

Style.—The architecture which prevailed in England during the reign dors; its period is generally restricted to the end of the reign of Henry

.—A small tower, especially at the angles of larger buildings, sometimes ing and built on corbels, and sometimes rising from the ground.

Order.—The plainest of the five orders of Classic architecture.

num.—The triangular recessed space enclosed by the cornice which pediment. The Greeks often placed sculptures representing subjects l with the purposes of the edifice in the tympana of temples, as at the n and Ægina.

Under-croft.—A vaulted chamber under ground.

Upset.—To thicken, and shorten as by hammering a heated bar of iron on the end.

Vagina.—The upper part of the shaft of a terminus, from which the bust or figure seems to rise.

Valley.—The internal angle formed by two inclined sides of a roof.

Valley Rafters.—Those which are disposed in the internal angle of a roof to form the valleys.

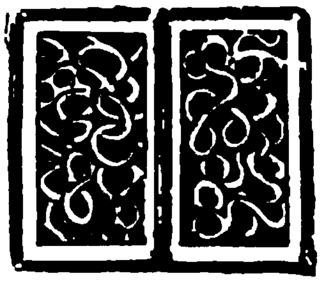
Vane.—The weathercock on a steeple. In early times it seems to have been of various forms, as dragons, etc.; but in the Tudor period the favorite design was a beast or bird sitting on a slender pedestal, and carrying an upright rod, on which a thin plate of metal is hung like a flag, ornamented in various ways.

Vault.—An arched ceiling or roof. A vault is, indeed, a laterally conjoined series of arches. The arch of a bridge is, strictly speaking, a vault. Intersecting vaults are said to be groined. See *Groined Vaulting* for fuller description of vaults.

Verge.—The edge of the tiling, slate or shingles, projecting over the gable of a roof, that on the horizontal portion being called eaves.

Verge Board.—Often corrupted into Barge Board; the board under the verge of gables, sometimes moulded, and often very richly carved, perforated, and cusped, and frequently having pendants, and sometimes finials, at the apex.

Vermiculated.—Stones, etc., worked so as to have the appearance of having been worked by worms.



VERMICULATED.

Vestibule.—An anti-hall, lobby, or porch.

Vestry.—A room adjoining a church, where the vestments of the minister are kept and parish meetings held. In American Protestant churches, the Sunday-school room is often called the vestry.

Viaduct.—A structure of considerable magnitude, and usually of masonry, for carrying a railway across a valley.

Vignette.—A running ornament, representing, as its name imports, a little vine, with branches, leaves, and grapes. It is common in the Tudor period, and runs or roves in a large hollow or casement. It is also called Trayle.

Villa.—A country house for the retreat of the rich.

Volute.—The convolved or spiral ornament which forms the characteristic of the Ionic capital. Volute, scroll, helix, and cauliculus are used indifferently for the angular horns of the Corinthian capital.

Voussoir.—One of the wedge-like stones which form an arch; the middle one is called the key-stone.

Wainscot.—The wooden lining of walls, generally in panels.

Wall Plates.—Pieces of timber which are placed on top of brick or stone walls so as to form the support to the roof of a building.

Warped.—Twisted out of shape by seasoning.

Water Table.—A slight projection of the lower masonry or brickwork on the outside of a wall a few feet above the ground as a protection against rain.

Weather Boarding.—Boards lapped over each other to prevent rain, etc., from passing through.

Weathering.—A slight fall on the top of cornices, window-sills, etc., to throw off the rain.

st.—A small door opening in a larger. They are common in mediæval nd were intended to admit single persons, and guard against sudden s.

.—A turn, a bend. A wall is *out of wind* when it is a perfectly flat

g.—A side building less than the main building.

es.—The partition between two chimney flues in the same stack.

ARCHITECTURAL TERMS AS DEFINED IN VARIOUS BUILDING LAWS,

COMPILED BY THE AMERICAN ARCHITECT AND BUILDING NEWS, PAGE 150, VOL. XXXIII.

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TERMS DEFINED.

[*The following terms chance to be defined in sundry building codes—which are mentioned in each case. The fact that other codes are not mentioned is not necessarily a proof that the term is not also elsewhere in use as defined.*]

Adjoining Owner.—The owner of the premises adjoining those on which work is doing or to be done. [*District of Columbia.*]

Alteration. Any change or addition except necessary repairs in, to, or upon any building affecting an external, party, or partition wall, chimney, floor, or stairway, and "to alter" means to make such change or addition. [*Boston and Denver.*]

Appendages. Dormer-windows, cornices, mouldings, bay-windows, towers, spires, ventilators, etc. [*Chicago, Minneapolis.*]

Areas. Sub-surface excavations adjacent to the building-line for lighting or ventilation of cellars or basements. [*District of Columbia.*]

Attic Story. A story situated either in whole or in part in the roof. [*Denver and District of Columbia.*]

Base. "The *base* of a brick wall" means the course immediately above the foundation wall. [*Cincinnati and Cleveland.*]

Basement Story. One whose floor is 12″ or more below the sidewalk, and whose height does not exceed 12′ in the clear; all such stories that exceed 12′ high shall be considered as first stories. [*Chicago, Louisville.*]

A story whose floor is 12″ or more below the grade of sidewalk. [*Milwaukee.*]

A story whose floor is 3′ or more below the sidewalk, and whose height does not exceed 11′ in the clear; all such stories that exceed 11′ high shall be considered as first stories. [*Minneapolis.*]

A story suitable for habitation, partially below the level of the adjoining street or ground.[1] [*District of Columbia and Denver.*]

(See **Cellar.**

Bay-window. A first-floor projection for a window other than a tower-projection or show window. [*District of Columbia.*]

Any projection for a window other than a show-window. [*Denver.*]

[1] And below the first floor of joists. [*District of Columbia.*]

g Walls.—Those on which beams, trusses, or girders rest. [*New York Francisco.*]

Building.—A building the walls of which are built of brick, stone, ›ther substantial and incombustible materials. [*Boston, Denver, and ;ity.*]

ıg.—Any construction within the scope and purview of these regula- *istrict of Columbia.*]

ıg Line.—The line of demarcation between public and private space. *of Columbia.*]

ıg Owner.—The owner of premises on which work is doing or to be *istrict of Columbia.*]

ss buildings shall embrace all buildings used principally for business thus including, among others, hotels, theatres, and office-buildings. *Louisville, Milwaukee, and Minneapolis.*]

—Basement or lower story of any building, of which one-half or more eight from the floor to the ceiling is below the level of the street[1] .[2] [*Boston, Denver, and Kansas City.*]

of building below first floor of joists, if partially or entirely below the ıe adjoining parking, street, or ground, and not suitable for habitation. *of Columbia.*]

t-mortar.—A proper proportion of cement and sand without the ad- ıf lime. [*Kansas City.*]

›n Wall.—One that separates part of any building from another part ne building. [*Cincinnati and Cleveland.*]

earing walls extending through buildings from front to rear, and sepa- ›res and tenements in buildings or blocks owned by the same party. *olis.*]

ırtition-wall.)

ng-house Class.—All buildings except public buildings and buildings rehouse class. [*Cincinnati and Cleveland.*]

ot apply to buildings accommodating more than three families. [*San* ›.]

ıal Wall.—Every outer wall or vertical enclosure of a building other rty-wall. [*Boston, Cincinnati, Cleveland, Denver, District of Columbia, ity, and Providence.*]

Story.—The story the floor of which is at or first above the level of the or adjoining ground, the other stories to be numbered in regular suc- ounting upward. [*Denver and District of Columbia.*]

g Course.—A projecting course or courses under base of foundation *incinnati and Cleveland.*]

ation.—That portion of wall below level of street curb,[3] and, where the t on a street, that portion of wall below the level of the highest ground ıe wall. [*Boston, Kansas City, New York, and Providence.*]

of exterior wall below surface of adjoining earth or pavement, and ›f partition or party wall below level of basement or cellar floor. *of Columbia and Denver.*]

ation, Basement, or Cellar Walls.—That part of walls of building that the floor or joists, which are on or next above the grade line. [*Detroit.*]

ıd. [*Providence.*]

ıot suitable for habitation. [*Denver.*]

serve as supports for piers, columns, girders, beams, or other walls." *k.*]

Portion of the wall below the level of street curb, in front of the central line of building. [*San Francisco.*]

Incombustible scantling partition.—One plastered on both sides upon iron lath or wire cloth, and filled in with brickwork 8″ high from floor, provided the building is not over 80′ high. [*Chicago.*]

Incombustible Roofing.—Covered with not less than three (3) thicknesses roofing-felt, and good coat of tar and gravel, or with tin, corrugated-iron, or other fire-resisting material with standing-seam or lap-joint. [*Denver.*]

Lengths. Walls are deemed to be divided into distinct *lengths* by return walls, and the length of every wall is measured from the centre of one return wall to the centre of another, provided that such return walls are external or party cross-walls of the thickness herein required, and bonded into the walls so deemed to be divided. [*Cincinnati and Cleveland.*]

Inflammable Material.—Dry goods, clothing, millinery, and the like in stores, flyings or goods in factories, or other substance readily ignited by droppings or flyings from electric lights. [*Minneapolis.*]

Lodging-house. A building in which persons are temporarily accommodated with sleeping[1] apartments, and includes hotels. [*Boston and Kansas City.*]

Any building or portion thereof in which persons are lodged for hire for less than a week at one time. [*District of Columbia and Providence.*]

Any building or portion thereof in which persons are lodged for hire temporarily, and includes hotels. [*Denver.*]

Mansard Roof.—One formed with an upper and under set of rafters, the upper set more inclined to the horizon than the lower set. [*Denver and District of Columbia.*]

Oriel Window.—A projection for a window above the first floor. [*District of Columbia.*]

Partition. An interior division constructed of iron, glass, wood, lath and plaster, or other destructible natures. [*District of Columbia.*]

Partition-wall.—Any interior wall of masonry in a building. [*Boston, Kansas City, and Providence.*]

An interior wall of non-combustible material. [*District of Columbia.*]

Any interior division constructed of iron, glass, wood, lath and plaster, or any combination of those materials. [*Denver.*]

(See **Division Wall.**)

Party-wall.—Every wall used, or built, in order to be used, as a separation of two or more buildings.[2] [*Boston, Cincinnati, Cleveland, Denver, Kansas City, and Providence.*]

A wall built upon dividing line between adjoining premises for their common use. [*District of Columbia.*]

Parking. The space between the sidewalk and the building line. [*District of Columbia.*]

Parking Line. The line separating parking and sidewalk. [*District of Columbia.*]

Public Building. Every building used as church, chapel, or other place of public worship; also every building used as a college, school, public hall, hospital, theatre, public concert room, public ball-room, public lecture-room, or for any public assemblage. [*Boston, Chicago, Cincinnati, Cleveland, Denver, Kansas City, and Minneapolis.*]

Such buildings as shall be owned and occupied for public purposes for this

[1] Sleeping apartments. [*Kansas City.*]

[2] To be used jointly by separate buildings. [*Cincinnati and Cleveland.*]

the United States, the corporation of the City of Brooklyn, or othe schools within said city. [*Brooklyn.*]

lic Hall.—Every theatre, opera-house, hall, church, school, or other build ended to be used for public assemblage. [*Milwaukee and Louisville.*]

urn Wall.—No wall subdividing any building shall be deemed a **returi** as before mentioned, unless it is two-thirds the height of the external o valls. [*Cincinnati and Cleveland.*]

l.—A skeleton structure for storage or shelter. [*District of Columbia.*] structure, enclosed only on one side and end, and erected on th .. [*San Francisco.*]

or closed board structure. [*Denver.*]

v-window.—A store-window in which goods are displayed for sale o sement. [*District of Columbia and Denver.*]

ire thereof.—The square or level of the walls before commencing th or roof. [*District of Columbia.*]

dard Depth for Foundations.—For brick and stone buildings, 14 curb line. [*San Francisco.*]

dard Depth of Cellars.—16′, measured down from sidewalk grade a y line. [*Memphis.*]

dard Iron Door.—Made of No. 12 plate-iron, frame or continuou ″ x $\frac{3}{8}$″ angle-iron, firmly riveted. Two panel doors, to have proper cross ne panel on either side, fastened together with hooks or proper bolts to ttom, and with not less than two lever-bars. All doors hung on iro of $\frac{3}{8}$″ x 4″ iron, securely bolted together through wall, swung on thre fitting close to frame all around ; sill between doors, iron, brick, or stone not less than two (2) inches above floor on each side of opening. Linte or, brick, iron, or stone. Floors of basement, when doors are to swing r cement, in no case wood. [*Denver.*]

dard Skylight.—Constructed of wrought-iron frames, with hammere -light glass not less than 1″ thick ; not larger than 10′ by 12′, except b permission of the Inspector. [*Denver.*]

ehouse.—(See **Warehouse Class.**)

et.—All streets, avenues, and public alleys. [*Minneapolis.*]

ment-house.—A building which, or any portion of which, is to be occu r is occupied, as a dwelling by more than three [1] families living independ f one another, and doing their cooking upon the premises. [*Boston , and Kansas City.*]

y more than two families [2] above the second floor, so living and cooking *and Kansas City.*]

ing which shall contain more than two rooms in front on each floor, o shall be built with a passage or arched way between distinct parts of th uilding, or which building shall be intended for the separate accommoda different families or occupants. [*Charleston.*]

tre.—Public hall containing movable scenery or fixed scenery whicl nade of metal, plaster, or other incombustible material. [*Chicago, Louis nd Milwaukee.*]

kness of a Wall.—The minimum thickness of such wall. [3] [*Boston nati, Cleveland, Kansas City, Milwaukee, and Providence.*]

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o instead of three. [*District of Columbia and Minneapolis.*]

n one floor, but having a common right in the halls, stairways, yards *Providence.*]

applied to solid walls. [... neapolis and Providence.]

Tinned Covered Fire-door.—Wood doors or shutters, double thickness of wood, cross or diagonal construction, covered on both sides and all edges with sheet-tin, joints securely clinched and nailed. [*Denver.*]

Tower Projection.—A projection designed for an ornamental door-entrance, for ornamental windows, or for buttresses. [*District of Columbia.*]

Vault.—An underground construction beneath parking or sidewalk. [*District of Columbia.*]

Veneered Building.—Frame structure, the walls covered above the sill by a 4″ wall of brick, instead of clapboards. [*Common understanding in Chicago, Milwaukee, and Minneapolis, but not defined by law.*]

Warehouse Class.—Buildings used for the storage of merchandise, manufactories in which machinery is operated, breweries, and distilleries. [*Cincinnati and St. Louis.*]

Width of buildings shall be computed by the way the beams are placed; the lengthwise of the beams shall be considered and taken to be the widthwise of the building. [*New York and San Francisco.*]

Wholesale store, or storehouse, shall embrace all buildings used (or intended to be used) exclusively for purpose of mercantile business or storage of goods. [*Chicago, Louisville, and Milwaukee.*]

Wooden Building.—A wooden or frame [1] building. [*Boston, Kansas City, and Minneapolis.*]

Any building of which an external or party wall is constructed in whole or in part of wood. [*Denver and District of Columbia.*]

Having more wood on the outside than that required for the door and window frames, doors, shutters, sash-porticos, and wooden steps, and all frame buildings or sheds, although the sides and ends are proposed to be covered with corrugated iron or other metal, shall be deemed a wooden building under this law. [*Charleston and Nashville.*]

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[1] Or veneered. [*Minneapolis.*]

ALI AL INDEX TO ADVEI SI S.

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[illegible]

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6" [illegible]

6 [illegible]